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Discussion of papers published in the April through June, 1960, JOURNALS appears in this issue, along with title page, errata, contents, and indexes for Proceeds V. 56, thereby completing this volume. Discussion of papers published in the July through September, 1959, JOURNALS appeared in March, 1960; discussion of papers published October through December, 1959, appeared in June 1960; discussion of papers published January through March, 1960, appeared in the Part 2, September, 1960, issue. Discussion pages in this issue are numbered continuously from the final page of the Part 2, September issue.

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ERRATA and ADDENDA

The following additions and corrections should be made in the indicated papers and reports of *Proceedings* V. 56, July 1959 through June 1960.

* * * *

Title No. 56-11 "General Elastic Analysis of Flat Slabs and Plates"

p. 135 — in the second sentence of the second paragraph, change "parameters" to parameter *s*."

p. 146 — in the first paragraph, third line below the equation for M_c, change "$\frac{1}{4}K_c$" to "$1/(4K_c)$."

* * * *

Current Reviews section

p. 184 — "Prestressed sheet piles enclose jetties." The article originally appeared in the *Engineering News-Record*, V. 162, No. 19, May 1959, pp. 59-60, 63 and not in *Civil Engineering*, V. 29, No. 5.

* * * *

Current Reviews section

p. 427 — "Contribution to the stability of cantilever girders." The expression "$I_c \geq I_b$" should read "$I_c >> I_b$."

p. 430 — "On the Analysis of Long Cylindrical Shells." The following information should be added: discussion by G. Röntsch was printed in *Der Bauingenieur*, Mar. 1959, pp. 113-116.

* * * *

Title No. 56-30 "Self-Service Parking Structures"

p. 485 — Acknowledgment should also be given to Nick Y. Yakovljevitch, who acted as structural consultant for Enco Engineering Co.

* * * *

Title No. 56-31 "Design of L-Shaped Columns with Small Eccentricities"

p. 489 — in the notation, in the expression for $r_{g\,min}$ change A_g to A_c. In the expression for X_g change b to B.

p. 494 — in the definitions above Eq. (20), the term $F_b = 0.45f_c'$ should be a separate item and not part of the expression for f'_{bg}. The term μ_g under "Concrete section," should read $\mu_g = -0.0122$.

p. 494 — in the last line the expression "$Q = 0.08$" should read "$Q = 0.8$".

p. 496 — the correct title in Reference 4 is "Spannungnachweise bei schiefer Biegung mit und ohne Laugskraft in belieiebigen Querschnitten."

* * * *

Title No. 56-36 "Design of Beams Subject to Torsion Related to the New Australian Code"

p. 600 — footnote should read: ". . . are obtained from the elastic and the plastic theory."

p. 603 — first expression in second equation on page should read:

$$M_{tv} = N\left(\frac{I}{\sqrt{2}} + \frac{C}{\sqrt{2}}\right) \ldots$$

p. 610 — equation for area of longitudinal steel should read:

$$A_{sl} = \frac{A_{sv}}{s}\,(X_1 + y_1)$$

* * * *

Title No. 56-39 "Stresses in Deep Beams"

p. 654 — under "Variation of stresses related to location of loads," paragraph (a) should read: ". . . in the outer thirds of the length. When load is applied only on the outer thirds of the top, tension is caused in the top fiber."

• • • •

Title No. 56-40 "Design of Prestressed Lift Slabs for Deflection Control"

p. 689 — The last line on the page which reads ". . . and live load is 0.30 f_c' . . ." should be ". . . and live load is 0.03 f_c' . . ."

• • • •

Title No. 56-41 "Behavior and Strength in Shear of Beams and Frames Without Web Reinforcement"

p. 697 — definition of $(N/V)_u$ should read: "ratio of axial load to shear at a particular . . ."

p. 699 — In the continued portion of Table 1, in the caption inserted at the head of the tabular data, the third line in this caption which reads: "Reinforcement: 3—#4 bars; $p = 1.01$ percent" should read: "Reinforcement: 2—#9 bars; $p = 3.36$ percent"

p. 704 — heading for sixth column of Table 2 should read: "Midspan steel strain* $\times 10^3$."

• • • •

Title 56-43 "Distribution of Torsion and Bending Moments in Connected Beams and Slabs"

p. 761 — in the third line under the heading "Special cases," change $x = \frac{1}{2}$ to read $x = l/2$. In the formula immediately following this line and which begins "$T = m_o$ . . ." change ". . . $= m_o \cdot \frac{1}{2}$" to read ". . . $= m_o \cdot l/2$".

p. 764 — change Eq. (11) to the following corrected form

$$\therefore \mu = \frac{k\,t\,l}{b^3}\left(\frac{t}{S}\right)^{\frac{1}{2}}$$

p. 764 — the line immediately under Eq. (11) which reads ". . . where K is determined . . ." should be ". . . where k is determined . . ."

p. 764 — in Table 2 the values 141 and 124 which are given for k should be changed to 1.41 and 1.24, respectively.

p. 765 — in the second equation from the top of the page, change the numerator so the equation reads

$$U = 0.08\,\frac{k\,l}{\sqrt{S}} = \ldots$$

p. 767 — in the third equation from the top of the page, change it to read

$$\therefore m = A \cosh cx + B \sinh cx$$

p. 767 — change Eq. (14) to read

$$T = -\frac{g\,S^2}{12\,c\,\cosh\frac{c\,l}{2}}\,\mathrm{Sin}\,cx$$

p. 772 — in the second equation from top of page, change ". . . $(bl/h)^2$" to read ". . . $(bl/d)^2$". First expression in Eq. (21) should read "$d^2m/dx^2 - a$ . . .".

• • • •

Current Reviews section

p. 896 — add the following to the first sentence of the review "Plastic buckling of the circular-cylindrical shell" so it reads—". . . the radial and axial directions; isotropic: identical moduli in the radial and axial directions)."

p. 897 — change the word in the first sentence of the review "Analytical stress determination for tunnel cross sections" from ". . . conformed . . ." to ". . . conformal . . ."

* * * *

Disc. 56-6 "Properties of Nuclear Shielding Concrete"

p. 924 — in Table A the saturated density is given in lb per cu yd; change this to read lb per cu ft. In the footnote under the same table change the word "regarded" to "regraded."

* * * *

Title No. 56-50 "Internal Forces in Uniformly Loaded Helicoidal Girders"

p. 1016 — there is a typographical omission in the equation given for δ_{zw}. Each term on the right-hand side of the equation should be multiplied by "sec α." In the actual computations for X_r and X_s it was included, so these values are correct as tabulated.

* * * *

Title No. 56-53 "Design of Unsymmetrical Reinforced Concrete Sections"

p. 1064 — in Eq. (11), the denominator should be changed from A_{j-d} to $A\,j\,d$.

p. 1067 — in the equation directly above Table 1, change

$$d = \frac{d\,I_{xy}}{j\,d\,s_s - I_{yy}} = \ldots$$

to

$$\propto d = \frac{d\,I_{xy}}{j\,d\,s_s - I_{yy}}$$

p. 1069 — in the second equation from the top of the page, change the number to read $jd = 8.94 + m = 10.815$ in. On the same page and two equations below the above mentioned equation, change the numbers to read

$$M = jdc = 10.815 \times 6320 = 68{,}500 \text{ in.-lb.}$$

* * * *

Current Reviews section

p. 1084-1085 — in "Damaging reactions in concrete," change the fourth sentence of the third paragraph to read, in part, ". . . for reactive aggregate, nor do admixtures, proved to be effective, work in this case."

* * * *

Title No. 56-60 "Elastic Analysis of Shear Walls in Tall Buildings"

p. 1213 — in Eq. (2) the denominator is given as $h\phi m$; it should read $h\phi_m$.

p. 1221 — in Table 2 the last two column heads at the right side of the table are given as V_{wi} and M_{gi}; these should read V_{wi} and M_{gi}.

* * * *

Title No. 56-61 "Method of Assessing Probable Fire Endurance of Load-Bearing Columns"

p. 1233 — the fourth line from the bottom which reads ". . . 1½ in. of neat gypsum plaster . . ." should be ". . . ½ in. of neat gypsum plaster . . ."

* * * *

Disc. 56-22 "Comparison of Measured and Calculated Stiffnesses for beams Reinforced in Tension Only"

p. 1353 — the first full line directly under Table F that reads: "The stiffness of cracked section is almost proportional to percentage of steel" — is actually the heading for Table F.

p. 1353 — in the second sentence of the second paragraph under the heading *First Example*, change that portion of the sentence which reads ". . . the curve near A is antimetric." to ". . . the curve near C is antimetric."

* * * *

Disc. 56-34 "Evaluation of Concrete and Mortar Mixes"

p. 1388 — in the tenth line change "0.50 ± 0.5" to "0.50 ± 0.05."

* * * *

Disc. 56-36 "Design of Beams Subject to Torsion Related to the New Australian Code"

pp. 1390-1391 — the two figures are mislabeled. Figure A should be labeled Fig. B, and vice-versa.

* * * *

Disc. 56-43 "Distribution of Torsion and Bending Moments in Connected Beams and Slabs"

p. 1435 — in the first line change "Eq. (24)" to "Eq. (37)."

p. 1436 — in the caption to Fig. H change "K_s" to "k_s."

p. 1437 — in the two equations at the bottom of the page change "K_s to k_s."

p. 1438 — in the first equation the term "$G/E \approx 0.43$" is not part of the equation for β.

p. 1438 — in the eighth line from the top change "K_{cs}" to "$K_{Columns}$."

p. 1438 — in the first, third and fifth lines under "Flexural stiffness of slab panels . . ." change "K_s" to "k_s."

p. 1438 — in the second line under the heading "Distribution Factors" change "K_s" to "k_s."

p. 1438 — in the fourth line under the heading "Distribution Factors" change "μ" to "$1 - \mu$."

p. 1439—in each of the equations under "at A" and "at B" change $\times$ to $\sim$.

p. 1440—in the last two lines change "M_A" to "m_A."

p. 1441—in the first line above the figure change "M_o" to "m_o."

p. 1441—in the fourth line change "M_o" to "m_o."

p. 1441—in Eq. (37) change "M_o/M_A" to "m_o/m_A" and "Fig. D" to "Fig. H."

p. 1441—in the fifth line above the figure change "M_B" to "m_B."

p. 1441—change the caption to Fig. K to read "Moment diagrams for slabs and beams."

p. 1442—in the fifth line change "M_B" to "m_B."

P. 1442—in the eighth line change "M_o" to "m_o."

p. 1442—in the tenth and eleventh lines changes "ΣM_B" and "ΣM_o," respectively, to "Σm_B" and "Σm_o," respectively.

p. 1446—in the equation for J, b^2 should be b^3.

* * * * *

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Discussion of a paper by Henry J. Cowan:

Design of Beams Subject to Torsion Related to the New Australian Code*

Discussions by Boris Bresler, Bruce H. Falconer, Hans Gesund, Thomas Paulay, and Paul Zia appeared in ACI JOURNAL, Part 2, Sept. 1960, pp. 1389-1400.

AUTHOR'S CLOSURE

The author would like to thank all those who have contributed to the discussion for their valuable comments. Most of the comments stress limitations in the torsional design method proposed. The author agrees with most of these; however, he would like to point out that little research has been carried out on the torsional strength of concrete by comparison with its strength in transverse shear, and it is therefore hardly possible to devise rules for torsional design which have not yet been achieved for transverse shear. In particular it seems premature to attempt the development of an ultimate strength theory for torsion until an acceptable theory has been derived for transverse shear.

In this connection Professor Bresler's comments are particularly interesting, and the author hopes that Professor Bresler will in due course give a more detailed account of the Russian work described. This would seem to be of quite recent date, since the formulas differ from those given in the "Instruction for the Design of Sections of Reinforced Concrete Structural Elements — Ultimate Strength Design, Ministry of Industrial Construction, Moscow 1956,"† and also from those given by Sachnowski in his textbook.‡

Professor Gesund's remarks are also essentially based on this point. As in the case of transverse shear, the stresses in the torsional shear reinforcement are small until the concrete cracks: this is confirmed by the author's strain measurements. They rise steeply when diagonal tension cracks form; but the contribution of the concrete to the torsional resistance moment never falls to a negligible proportion. Until the controversy "European *versus* American design for shear reinforcement" is resolved, there is likely to be a difference between torsional rules in the "European-style" (i.e., German) and the "American-style"

*ACI JOURNAL, V. 31, No. 7, Jan. 1960 (*Proceedings* V. 56), p. 591. Disc. 56-36 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 6, Part 2, Dec. 1960 (*Proceedings* V. 56).

†Translated by Portland Cement Association, Chicago, p. 68.

‡German translation: Sachnowski, K. W., *Stahlbetonkonstruktione*, VEB Verlag Technik, Berlin, p. 32.

(i.e., Australian) codes. The truth probably lies somewhere between these two extremes.

The reasons for allowing the same steel stress in torsion as in flexural tension, and for limiting the concrete stress to 360 psi are that in the Australian (and the ACI) code this also applies to transverse shear.

There is some disagreement between experimental data on the angle of inclination of the cracks in combined bending and torsion. The author throughout found cracks at approximately 45 deg in reinforced beams subjected to combined torsion and bending, with another series of near-vertical cracks due to bending superimposed. Nylander obtained a similar result with reinforced beams, but Bresler and Pister obtained differing inclinations of the cracks with plain beams. The evidence is hardly sufficient to warrant a firm conclusion, but it may be accounted for by the presence or absence of shear reinforcement.

Mr. Zia is correct in stating that the most highly stressed longitudinal bars would be in the middle of the sides. However, the Australian Code was primarily concerned with producing some simple and generally acceptable rules where hitherto there had been none. Bars in the middle of the sides without corner bars might strike the contractor as a little odd, and they would fail to give good anchorage to the stirrups — a most important consideration. In actual fact strain measurements by the author show that, probably due to some redistribution of stress, the stresses in corner bars are not negligible at high loads.

The rules for the maximum spacing of torsional shear reinforcement would be the same as for transverse shear reinforcement.

Some of Mr. Falconer's comments may be answered in the same way. There is evidently not much difference between the theory to which he refers, based on a close approximation, and the author's precise elastic solution; however, both would require a table or a lengthy equation for their evaluation. For the sake of simplicity the Australian code used a single constant, corresponding to the square, which for elongated rectangles is a little uneconomical. When torsion is a secondary effect this is usually of no consequence. The Australian code states that the formula given may be used in the absence of more precise calculations, which would, however, be indicated for large torsional moments.

As has already been stated, the limiting stresses for torsion are the same as for transverse shear, except that the somewhat severe and complicated conditions for transverse shear reinforcement in the range $180 \leq v \leq 360$ psi have been omitted. The measured stresses in the torsional reinforcement, particularly in the longitudinal reinforcement are rarely high, and it is quite common for beams to fail without any of the torsional reinforcement yielding. Consequently an increase in the amount of longitudinal steel would probably serve little purpose.

The reasons for departing from the transverse shear rules are (1) that it is difficult to think of a satisfactory way of combining two separate types of torsional shear reinforcement, and (2) the concrete core remains uncracked in torsion, while in transverse shear the diagonal crack usually runs right across the face.

The author agrees with Mr. Paulay that for large beams the longitudinal steel should be distributed around the circumference. He does not agree with his assumption, also made by Rausch, that the steel stress in a leg is proportional to the distance from the center of the beam. Actually the reverse is true, since the maximum stress occurs at the middle of the sides, and the minimum at the corners. A parabolic distribution of stress over the sides of the stirrups would give a reasonable approximation.

The formulas given in the Australian code are essentially intended for secondary torsion ("In the absence of more precise calculations, the maximum stress may be computed from the following formulae, . . . the quantity of reinforcement may be computed from the following formulae"). Bearing in mind that secondary torsion has been ignored in the past, it was felt that elaborate rules would be disregarded by designers and building authorities alike; however, there is clearly room for much improvement.

Discussion of a report by ACI Committee 609:

Consolidation of Concrete*

By T. N. W. AKROYD, V. M. MALHOTRA, PAUL L. OSWEILER, and M. SPINDEL

By T. N. W. AKROYD†

This is a most interesting and valuable report, marred only by its title. The committee has called a process usually known as compaction, that is the process of removing air voids from concrete after deposition, by the misleading name consolidation.

To those of us who are interested in soils and soil testing as well as concrete, compaction is a process very different from consolidation. The compaction of a soil is analogous to the compaction of concrete but in concrete there is no real counterpart to the consolidation of soil. Since knowledge of both soils and concrete are basic requirements of an engineer, let us make life a little easier by using our technical jargon consistently to mean the same thing or the same kind of process.

In this respect I notice that nowhere in the bibliography does any writer refer to consolidation although a number do refer to compaction; indeed, the work of Meissner on which the committee says the report is based, is entitled "Compacting Concrete by Vibration."

Turning now to the body of the report I should like to refer particularly to the section on heavy aggregates. This section of the report (Section 208) gives what I believe to be a false impression of the use of heavy aggregates in that it appears to indicate difficulties which in fact do not appear in practice.

Experience over the last few years with heavy aggregates to produce dry densities of 240 lb per cu ft has indicated that traditional methods of placing and compaction can produce heavy and stronger concrete than with prepacked heavy aggregates.

If possible, the coarse and fine aggregates should have the same specific gravity to avoid differential segregation during compaction. From a series of tests, including full scale tests, it was found that high grade magnetite is a most suitable aggregate.

The characteristics of a magnetite mix are somewhat peculiar. Using a 1:2:8 (by weight, cement: fines 3/16 in. to dust: coarse magnetite 1½ to 3/16 in.); and a w/c ratio of 0.45 by weight, the concrete in the

*ACI JOURNAL, V. 31, No. 10, Apr. 1960 (*Proceedings* V. 56), p. 985. Disc. 56-49 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 6, Part 2, Dec. 1960 (*Proceedings* V. 56).

†Chief Engineer, Constructional Services Limited, London, England.

mixer or skip had the appearance of a heap of dryish stones. It flowed readily using a 2½-in. internal vibrator but the effective radius of the vibrator which is normally 3 ft was reduced to 18 in. No segregation occurred and full compaction was achieved, while in addition the mix could be dropped 6 to 8 ft without segregation. The workability would be classified as "low" while the Vee Bee consistometer gave readings between 4 and 6 sec. The strengths at 28 days varied from 6500 to 7000 psi.

The use of this concrete in a mass heavily reinforced and containing pipes and large conduits produced no difficulties while on the other hand the same aggregate in a prepacked form resulted in lower strengths and densities due in some measure to the necessary fluidity of the heavy grout and the coating of fine dust around the crushed aggregate.

The advocation by the committee of prepacked concrete for placing in complex forms is completely contrary to our experience.

By V. M. MALHOTRA*

The committee is to be congratulated for emphasizing the need for vibration in modern concrete practice. On a recent job in southern India the writer was engaged in producing precast reinforced concrete beams with a 4 ft deep I-section and having a length of about 50 ft. The concrete was to be designed to give a minimum compressive strength of 5000 psi at 28 days. The concrete was to be placed in a hot temperature (89 to 93 F). It was most desirable to keep the water content per cubic yard of concrete to the absolute minimum to avoid excessive shrinkage and to keep cement costs down. After a series of trial mixes it was found that a concrete mix with a w/c ratio of 0.42 having a slump of 1½ in. at the mixer could be used without adverse effects. The water content of this mix was 310 lb per cu yd. Form vibrators as well as immersion vibrators were to be used.

During the casting operation it was found that in spite of the best efforts of the supervisors, the inexperienced casting crew had the habit of leaving vibrators at one place for far too long a time. To overcome this problem of over-vibration, the writer reduced the water content to 298 lb per cu yd to give a slump of ½ in. and less at the mixer. Making full use of the "bad habit" of the casting crew, satisfactory beams were cast with a net saving of about 30 lb of cement per cu yd of concrete.

The committee has used the word "melting down" the concrete; the writer feels that perhaps "dispersing" could have been more appropriate.

*Member American Concrete Institute, Concrete Engineer, Engineering Construction Corp., Ltd., Madras, India.

By PAUL L. OSWEILER*

The following comments apply particularly to Section 209(c) of the report.

The speeds indicated for centrifugally cast pipe are generally too low. The pipe manufacturer tries to operate at as high a speed as possible. In general speeds are of the order of 3000 ft per min or higher, in some cases as high as 5000 or more ft per min. High velocity results in dense concrete with a much faster time cycle on and off the machines.

The discussion of consolidating concrete with form vibrators only partially covers the field. It is true that high frequency vibration is considered best for concrete with 2- to 4-in. slump. However, new methods being developed by the industry use approximately 3400 rpm vibration with very high amplitude. The concrete placed with these systems is a zero slump concrete, similar to that used in the tamp pipe mixes.

In general the practice of dumping a large volume of concrete in thick layers traps the air and does not result in increased production rates. It has been found that continuous but slow feeding produces the best texture in the same time period. Therefore, best results with the fewest air bubbles can be obtained where the concrete is slowly vibrated into the mold. This permits the material to build up in 2- to 3-in. layers and allows the trapped air to escape more freely.

A process for making pipe not mentioned in the above methods consists of the roller suspension process where a zero slump concrete is placed in a mold while the mold is suspended and rotated on a heavy shaft. This supporting shaft rolls the dry mix into place and produces a pipe quite similar to that made by the process that uses centrifugal force, vibration, and rolling.

By M. SPINDEL†

This report gives excellent information not only about consolidation of concrete but also on the most suitable concrete mixes for the different types of concrete.

The writer, with reference to his discussion of a paper by Kaplan,‡ may be allowed some supplementary remarks which might be of some use in the designing, making, and testing of concrete.

First of all we always have to consider "fresh" concrete to consist of four materials: cement, sand and coarse aggregates, water, and air.

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*Member American Concrete Institute, Director of Research and Engineering, Price Brothers Co., Dayton, Ohio.

†Member American Concrete Institute, Research Engineer and Consultant, London, England.

‡Spindel, M., discussion of "Effects of Incomplete Consolidation on Compressive and Flexural Strength, Ultrasonic Pulse Velocity, and Dynamic Modulus of Elasticity of Concrete," ACI JOURNAL, V. 32, No. 3, Part 2, Sept. 1960 (*Proceedings* V. 56), p. 1463.

As to the latest investigations by competent scientists, it is believed that for the hydration of the cement particles not only the chemically combined amount of water for hydration is necessary, but also some physically absorbed water, i.e., together about 40 percent water of the weight of cement is needed for complete hydration. The rest of the water added remains as water holes, which, after drying, may become air voids. Both these water and air voids calculated by volume percentage reduce the strength, impermeability, and durability of concrete for the same type and quantity of cement and aggregates.

Therefore, in designing the concrete for various purposes the percentages of cement, aggregates, water, and air have to be proportioned in absolute volumes beforehand. This can easily be done with the "fourmatter parallelogram" which is used for designing the four main components of cement and the four components of concrete as well. For concrete, its main components in freshly made consolidated concrete can easily be calculated and the air tested with the air meters.

The advice given in the paper with regard to the importance of using the largest size possible for the aggregates, and the lowest possible workability, especially with regard to water-cement ratio, and using effective vibration for consolidation as far as necessary, is the best which could be given to concrete makers.

In addition two questions may be discussed more deeply:

1. Even the best mix proportions for cement, aggregate, water, and air and the best type of consolidation will not succeed in getting the results desired if the grading of the aggregates plus cement is not quite suitable to get the desired density of the concrete. For several decades special formulas and grading curves have been developed for aggregates alone or together with cement, both for continuous- or gap-grading, but the results on the job were not always quite according to expectations.

Then, apart from the slump test, special apparatus were built to test the workability and density of concrete. The latest of them is by vibrating the test mix in the laboratory.

As the writer pointed out in a paper,* the vibrating curves he obtained from published results by Dr. Humm of vibrating concrete without and with an admixture, showed graphically the work done by vibrating from the beginning to the end as a curve in the fourmatter parallelogram. Since then, the writer and other concrete testers have devised special apparatus for testing the consolidation of concrete by vibrating and obtaining by recording the settlement and work done by the vibrator which gives results suitable for every comparison of concrete mixes with regard to consolidation and workability. In this way the beneficial effect of admixtures to concrete could easily

*Spindel, M., *Beton und Eisen* (Berlin), 1935.

be demonstrated on recording drums and was done so for some years when the large dams in the Alps of Austria were built and some other concrete structures of importance were designed and constructed.

From the above experience it seemed to be clear that apart from the mix proportions and the size, shape, and smoothness of the aggregate, the workability of concrete depended mainly on the water-cement ratio and whether or not suitable admixtures were used to reduce the water-cement ratio or to entrain air or a combination of both.

About the benefit of the reduction of water-cement ratio, nothing more can be added to what was said in the paper and is already known.

With regard to the voids caused by air-entraining agents, some remarks may be allowed to point out its advantages and how disadvantages can be avoided.

As may be remembered, the writer was one of the first to agree to the beneficial effect of air entrainment in concrete, but at the very beginning the writer pointed out* that the amount of air entrained had to be limited in the same way as the water-cement ratio, i.e., that instead of the water-cement ratio, the water plus air-cement ratio had to be used as the basis.

This was in full agreement with test results obtained later by Gonnerman* who pointed out that the effect of air entrained voids had to be considered like that of ordinary voids with regard to reduction of strength. It is now fully understood that the necessary percentage of air entrained is dependent on the largest size of the aggregate, as the air is only in the mortar and only there successful. For many purposes entrained air between 3-4 percent will do, thus avoiding too much harm to strength where it is necessary to have it.

2. The other principle pointed out by the writer,† i.e., that "shape, size, and distribution of air pores should be tested by microscopic examinations and appropriate absorption tests," has resulted in some different types of microscopical tests published mainly in the ACI JOURNAL, but still it appears not to be taken very seriously, even in testing laboratories.

Therefore, the writer wishes to point out the risks of unfavorable size, shape, and distribution of voids. An example case can be illustrated with a piece of a broken bearing of bronze, consisting mainly of copper and tin and a few percentages of zinc plus lead. The latter was added mainly to reduce the friction and abrasion of the bearing. The lead in this bronze apparently had to play a role similar to that of

*Spindel, M., discussion of "The Function of Entrained Air in Concrete," ACI JOURNAL, V. 15, No. 2, Supplement, No. 1943 (*Proceedings* V. 39), p. 544-1.

*Gonnerman, H. F., "Tests of Concretes Containing Air-Entraining Portland Cements or Air-Entraining Materials Added to Batch at Mixer," ACI JOURNAL, V. 15, No. 6, June 1944 (*Proceedings* V. 40), pp. 477-506.

†Spindel, M., and Quinn, R. T., *Transactions*, Society of Engineers (London), Dec. 1945.

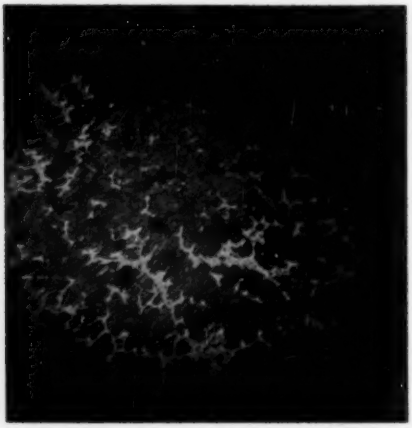

Fig. A — Magnification approximately 130 × showing black cavities of lead in the white and grey particles of bronze made of copper and tin; i.e., white is the chemical composition Cu_3Sn of copper and tin, the grey is the alloy of copper and tin

entrained air in concrete, i.e., added with the intention of giving small spherical particles of lead helping the lubrication in the bearing without harm to strength.

As may be seen from Fig. A, the black holes in particles of different composition, seen as white and grey, were not at all evenly distributed and neither very small nor spherical. These black voids in certain parts show sharp cornered shapes and continuing from one to another, thus giving the opportunity of developing cracks and breaking the bearing, especially when used under tensile strain even locally.

Soon after having been brought into use, in about 1930, this bearing was broken and we had to find at the Railway Material Research Station, the cause of failure, which, after microscopic tests, was stated to be due to the sharp cornered voids of too large a size accumulated in some places. Similar damage due to sharp cornered edges of holes was caused in rails, tires, etc. made of steel, and especially in welded joints where sharp corners have to be avoided.

The above examples were mentioned here only to explain the beneficial role of air entrained in concrete. The main advantage of air entrainment is to convert the harmful role of sharp cornered shape, large size, and bad distribution of air voids into the useful role of small air bubbles, thus avoiding harmful cracks and other disadvantages.

From the above it may be seen that high quality concrete for structures has to be dealt with in the laboratory in a similar way as high quality steels and metals, i.e., by microscopic examinations.

Discussion of a paper by A. C. Scordelis:

Internal Forces in Uniformly Loaded Helicoidal Girders*

By VICTOR R. BERGMAN, H. S. GEDIZLI, HELGE HARBOE, and AUTHOR

By VICTOR R. BERGMAN†

One deterrent to the use in America of helicoidal staircases has been the necessity, prior to the publication of the paper under discussion, for the designing engineer to plough through long and tedious computations in order to effect a so-called "exact solution." True, Hunziker[8] and Fuchssteiner[2] had both published in Europe expositions of the exact theory, and Gedizli[3] had published coefficients serving to simplify somewhat application of the Fuchssteiner equations, but their material, published in German, was neither generally known nor readily available to American engineers.

As a result, the American engineer, confronted with the task of designing such a structure and realizing that the work involved in an exact solution was excessive in relation to the possible savings in materials, often resorted to an approximate solution such as that suggested in the author's Reference 1. In 1957, Holmes published a paper[5] dealing with the theory of the helicoidal staircase and presenting equations for the computation of the two midspan redundants for the usual design case of the uniformly loaded girder. The present paper, by providing coefficients for the direct determination of the two midspan redundants, reduces the problem in effect to a statically determinate one, and laudably minimizes both the drudgery and the chance of making mistakes.

Another deterrent to the use of the exact theory has been the realization that the exactitude of the results may be open to serious doubts. Such doubts stem principally from the fact that the helicoidal staircase is a sharply curved member for which bending and torsional rigidities are customarily computed as though the member were straight and they were concentrated along the longitudinal axis.

Some of these doubts have been partially dispelled by Young and Scordelis in a recent study[6] of laboratory tests which enabled them to conclude that: "For girders with rectangular cross sections having a width-depth ratio between 1 and 16 and a ratio of center line radius to width between 3 and infinity, sufficiently accurate results for the end

*ACI Journal, V. 31, No. 10, Apr. 1960 (*Proceedings* V. 56), p. 1013. Disc. 56-50 is a part of copyrighted Journal of the American Concrete Institute, V. 32 No. 6, Part 2, Dec. 1960 (*Proceedings* V. 56).

†Vice-President, Godwin Construction Co., New York, N. Y.

TABLE A — EXAMPLES OF HELICOIDAL STAIRCASES
Conventional reinforced concrete construction unless otherwise noted

Location	Central angle; radii; slab thickness	Loading	Reinforcement	Designers	Source of information
Hospital at Cruces, Baracaldo, Bilbao, Spain	Approximately 240 deg $R_i = 3$ ft 2½ in.; $R_o = 9$ ft 9½ in. Thickness (normal to slab) = 16 in.		Bottom longitudinal: 11—⅞ in. diameter continuous; Top longitudinal: 6—⅞ in. diameter continuous plus 16—⅞ in. diameter (additional) at supports. Ties: ½ in. diameter at 4 in. on center at ends, and 5/16 in. diameter at 4 in. over remainder, plus 5/16 in. diameter "links" at 4 in. throughout	M. J. Marcide, Architect; C. Monzón y Reparaz, consulting engineer	*Concrete & Constructional Engineering* (London), Oct. 1950
Tidewater Inn, Easton, Md.	Approximately 210 deg $R_i = 5$ ft 9 in.; $R_o = 9$ ft 6 in. Normal thickness = 8 in.		Longitudinal: 1 in. diameter at 7 in. on center for top, and 5 in. on center for bottom. Ties: ½ in. diameter at 6 in. on center, measured along center line of stairs. Slab haunched at ends	Alfred Hopkins & Associates, Architect; I. Hellerman, consulting engineer	Personal communication
American Tobacco Co., Copenhagen, Denmark	Apparently 180 deg Thickness, measured vertically, 7½ to 10 in.	DL = 130 psf LL = 75 psf TL = 205 psf	Longitudinal bars: ½ in. diameter. Ties: ¼ in. diameter at 6 in. on center, measured along center line of stairs	Hans Rahiff, architect, Rassmussen & Schiotz.	*Civil Engineering*, May 1952
San Jose High School, San Jose, Calif.	150 deg $R_i = 6$ ft; $R_o = 12$ ft Normal thickness = 10 in.	DL = 175 psf LL = 75 psf TL = 250 psf	Longitudinal 1 in. diameter: 7 continuous, top and bottom, plus 3 top at each support. Ties: ⅜ in. diameter at 9 in. on center, measured along center line of stairs	E. J. Kump, architect; E. Moorehead, engineer, A. C. Scordelis, structural design	*Civil Engineering*, Nov., 1951
Royal Festival Hall, London, England	Apparently 180 deg Helical beam, 15 in. x 18 in.; cantilevered treads			Scott & Wilson, engineers	*Civil Engineering*, Feb. 1952

reactions to be used in design may be obtained by analyzing only the elastic line defined by the longitudinal centroidal axis of the girder, thus eliminating slab effects."

However, it should be observed that Young and Scordelis, acting with scientific propriety, carefully limited the range of validity of their findings to girders with a "ratio of center line radius to width between 3 and infinity." Now, an examination of Table A reveals that in actual helicoidal staircases the ratio of centerline radius to width of slab is as low as 1½ or even 1. The question naturally arises: Is it safe to assume that the conclusion quoted will apply also to slabs of the low ratios obtaining in actual practice?

The tests made by Young and Scordelis rather convincingly demonstrated the validity of results obtained by the application of the ordinary simplifying assumptions for ratios of center line radius ranging

TABLE A (cont.) — EXAMPLES OF HELICOIDAL STAIRCASES
Conventional reinforced concrete construction unless otherwise noted

Location	Central angle; radii; slab thickness	Loading	Reinforcement	Designers	Source of information
General Motors Continental Building, Antwerp, Belgium	316 deg 30 min R_i = 39.4 in.; R_o = 114.2 in. Normal thickness 10.2 in. at inner edge; 12.6 in. at outer edge	DL = 176 psf LL = 104 psf TL = 280 psf	Prestressed concrete design	Cole & DeRoeck architects; G. Magnel, engineer	*Civil Engineering*, Sept. 1951
Supreme Court Building, San Juan, Puerto Rico	220 deg R_i = 5 ft 6 in.; R_o = 10 ft 10 in. Thickness (normal to slab) = 10 in.		3—1 in. diameter each side (1 top, 1 bottom, 1 middle). Other top bars, 4—⅝ in. diameter; other bottom bars, 19—⅝ in. diameter, all continuous. Ties: ⅝ in. diameter at 4 in. on center at supports, increasing to 12 in. at midspan	Toro-Ferrer, architects, Puerto Rico; Charles H. Warner, Jr., consulting architect, New York; Harold E. Leeds, design consultant	Personal communication. Also *Architectural Forum*, Aug. 1957
Administration Building of La Suisse (insurance company) Lausanne, Switzerland	240 deg R_i = 1.08 m; R_o = 2.78 m Thickness (normal to slab) = 21 cm	2.3 ton per m (along horizontal projection of the longitudinal center line)	13—14 mm diameter, top and bottom, full length 4—14 mm diameter, top and bottom, at each support 3—14 mm diameter, top at outer edge 50—10 mm diameter twin hoops, each in two pieces to facilitate placing of top longitudinals. 8 spacings of 10 cm at each support; intermediate spacing = 15 cm on center	H. B. de Cerenville, engineer, Armin Hunziker, Structural designer	Personal communication

from infinity down to 3, and it might appear that no important error would be introduced if the range of validity were arbitrarily extended to include ratios of 1½ or even 1, such as occur in practice. However, the writer does not feel that it is quite safe to draw such a conclusion without substantiating evidence.

One obvious consideration concerns the quantities I and J, both of which figure importantly in the equations employed in Professor Scordelis' paper, and values for which are regularly calculated by formulas derived specifically for *straight* members. Their unmodified use in problems dealing with curved members is entirely satisfactory only when the curvature is not excessively pronounced. Moreover, Timoshenko[9] states that: "When the cross-sectional dimensions of a curved bar are not small in comparison with the radius of the center line, not only the strain energy due to bending moment but also that due to longitudinal and shearing forces must be taken into account."

The curved beam of Fig. A has been analyzed by Timoshenko, taking into account the strain energy due to bending moment and to longitudinal and shearing forces. He arrived at the following expression for the vertical deflection at the free end:[9]

$$\delta = \frac{\pi PR}{4AE}\left(\frac{12R^2}{h^2} + 2.12\right)$$

wherein A is the cross-sectional area and E/G is 2.6. It is readily seen that the second term in the parentheses, which represents the influence of the longitudinal and shearing forces, is practically negligible for $R/h \geq 3$. However, when the equation is examined for $R/h = 1$, the second term in the parentheses is found to equal almost 18 percent of the first term and can no longer be considered insignificant if one aspires to an exact solution.

The foregoing example may serve to give a rough idea of the possible errors involved in applying the assumptions used by Young and Scordelis beyond the range of validity they carefully defined. Actually, the helicoid is a far more complex structure than the curved beam analyzed by Timoshenko, and further experimentation seems desirable to establish the order of error involved when the type of theoretical analysis employed by Professor Scordelis in the preparation of the present paper is applied to helicoids of the sharp curvature sometimes encountered in practice.

Still another disturbing element in the application of exact theory to the analysis of a helicoidal stair slab arises from a constructional feature of the stair itself. In the construction of almost all staircases, the steps are cast integrally with the slab, producing a "sawtooth profile." What is the proper value of slab depth to use in computing the bending and torsional stiffnesses of the section? Fig. 4 giving torsion constants for rectangular sections follows the customary procedure in designating the thickness to be used as the "minimum dimension." It will be recognized, however, that the actually greater average depth due to the presence of the steps must inevitably modify the stiffnesses in both bending and torsion, although not necessarily to the same extent. The modifications may well be important because, as stated by the author on p. 1024: "A change in the b/h value has a

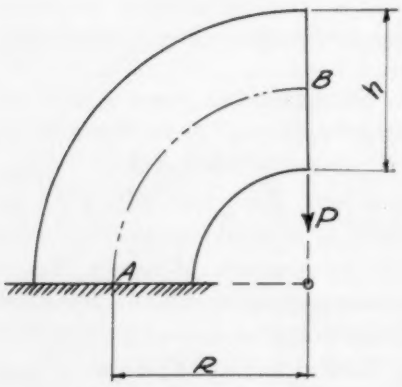

Fig. A—Curved beam

tremendous effect on maximum values of M_r and M_t in helicoidal girders having $\alpha = 10$ deg to $\alpha = 30$ deg. M_r and M_t are much smaller for $b/h = 16$ than $b/h = 1$."

The tests reported by Young and Scordelis[6] were conducted on helicoidal slabs of constant cross section, i.e., sans steps. A much simpler series of tests performed on straight slabs, some with and some without steps, would probably suffice to determine with sufficient accuracy the influence of the stepping on the bending and torsional stiffnesses. Until such studies have been made, it may be advisable for the analyst to estimate the possible range of variation in stiffnesses for a particular stair slab and to make a sufficient number of analyses for safety in design. The wide coverage of "redundants at midspan," provided by the author in his paper, is admirably suited to render such a study feasible for even the busier engineer.

Incidentally, among the many thousands of readers of the ACI JOURNAL, there surely must be engineers who have participated in actual loading tests on full-scale helicoidal staircases designed by the exact theory. It is to be hoped that anyone possessing such test results will submit them to ACI for publication. The writer regrets that he has no such data to offer, although he did receive last year the following information from Armin Hunziker, engineer, Lausanne, Switzerland (regarding a staircase described in Table A): "L'escalier a été construit en 1952 et il se comport très bien car jusqu'à ce jour on n'a pu constater aucune fissure," which may be translated as, "The staircase was built in 1952 and is behaving very well since to date no cracking has been detected."

In closing, the writer wishes to express his appreciation of this helpful paper which should prove of considerable practical assistance to anyone dealing with a problem of the type covered. The writer's discussion has not been intended to detract in the least from the worth of Professor Scordelis' work, but only to warn the possibly unwary reader not to apply the tabulated values in the unquestioning manner of the "handbook engineer."

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8. Hunziker, Armin, "Freitragende Wendeltreppen mit starrer Einspannung und horizontale Kreisringträger," *Schweizerische Bauzeitung*, Mar. 27, 1954, pp. 167-171.

9. Timoshenko, S., *Strength of Materials, Part II*, D. Van Nostrand Co., Inc., New York, 2nd Edition, pp. 83-84.

By H. S. GEDIZLI*

The writer agrees with the detailed tables computed by Professor Scordelis and congratulates him for his help to engineers who will be involved with the analysis of helicoidal stairs.

The writer hopes that it will be useful to call attention to a recent publication by Menn† in which the structural problems of a helicoidal girder of a helicoidal shell are completely solved, including their edge perturbations or edge conditions (in German: *Randstoerungen*). In fact, many problems in helicoidal stairs require shell solutions; continuity in shell or girder ends, i.e., the fully fixed supporting and girder conditions of helicoidal stairs, is rarely found in practice, so in the writer's opinion the tabulated results of Menn's theory will be welcomed also by designers.

By HELGE HARBOE‡

The author has presented an interesting and valuable paper for a helicoidal girder uniformly loaded and fixed at its ends, giving concise formulas for the two redundants at midspan and further, based on these, has attached a copious table containing numerical figures for the redundants for a great number of cases. This table, in itself quite unique, will enormously facilitate work for the practical engineer.

The writer has taken the liberty to check all the formulas for the unit moments m as well as for the final values δ of the relative displacements. Further he has in several cases checked the numerical values given for X_r and X_s in Table 1 and found them correct and would recommend that other readers interested in statics do the same as it presents a useful exercise for the "nonexpert," facilitated by the fact that the integrals involved can be solved using formulas found in handbooks (for instance, *Hütte* No. 1). The fact that the reader is presented with numerical figures for the redundants, avoids, as the author points out, lengthy and complex calculations, which even using a slide rule (not to be recommended) are tedious and easily subject to mistakes.

Also, the graphs given for the maximum bending and torsional moments for different values of ϕ, α and b/h are useful. However, the author might have mentioned where these moments act, which appears always to be at the supports, up to certain values of $\phi > 90$ deg.

In summary, it might be expressed that the paper inspired complete confidence to the writer (which is not generally the case with printed matters), not the least being the fact that a digital computer was used for calculating the numerical values of the redundants.

Regarding the formulas presented it might be permitted to make a

*Civil Engineer, Ankara, Turkey.

†Menn, C., "Kreisringtraeger und Wendelflaeche," *Mitteilungen* No. 30, Institut fuer Baustatik (ETH), Zurich, 1956.

‡Member American Concrete Institute, Civil Engineer, Rio de Janeiro, Brazil.

few observations. The integrals for δ_{xw}, etc. given in the appendix (not taking into consideration the minor influence of the normal forces nor the usually moderate influence of the eccentricity of the vertical load with regard to the center line of the girder) are developed only over half of the span from $\theta = 0$ to $\theta = \phi$ and should, therefore, doubtless receive the factor 2. Further in the base formula for δ_{xw} on p. 1016, the factor "sec α" is omitted (which, however, is taken into consideration in the formulas in the appendix) expressing that the integration should be carried out along the inclined center line of the girder. These omissions however do not alter the values of the redundants X_r and X_z.

Finally with regard to the remaining four redundants, which are equal to zero, it seems that the statement by the author: "using principles of symmetry, all but two of the redundants become equal to zero" might have been explained in a little more detail, perhaps as suggested by the following: Suppose that the normal force at midspan $+ X_n$ acts toward the supports, then an applied load of $- w$ will produce a normal force $- X_n$. By turning the helicoidal 180 deg round a horizontal axis parallel with the supports and situated at midspan the vertical load will again be acting downward, and it will be required that $+ X_n = - X_n$ or $X_n = 0$. Analog considerations can be used to prove that the other three remaining redundants also are equal to zero.

AUTHOR'S CLOSURE

The author appreciates the interest shown by the various writers who have discussed this paper and welcomes the opportunity of answering the questions raised.

The proper *design* of any reinforced concrete structure is a complex problem involving sound engineering analysis and judgment. In most cases assumptions must be made with respect to the elasticity and homogenity; variations in bending and torsional stiffnesses; support conditions; and general behavior of the structure. Once these assumptions have been made the resulting structure (sometimes called a mathematical model) is analyzed to determine the magnitude and distribution of the internal forces in the structure. While these results may be "exact" for the mathematical model, they can only be as accurate as the assumption made when referred to the actual structure.

The designer, therefore, when using the results of this analysis for determining cross section dimensions and reinforcement requirements must use sound engineering judgment in applying proper factors of safety based on his knowledge, first, of how closely the assumptions made for the mathematical model are approximated in the actual structure; and second, of the effect variations in each of the assumptions made will have on the analytical results obtained. The *design* of a helicoidal girder is no exception to the above pattern.

The paper under discussion was concerned with the *analytical* deter-

mination of the internal forces in a uniformly loaded helicoidal girder, fixed at its ends, with specified bending and torsional stiffnesses along its longitudinal axis. The tabulated results given are comprehensive enough so that at least this portion of the total problem of designing helicoidal girders can be removed from further study and attention may be turned to the many other facets of this problem which need careful consideration.

Since publication of the paper the author has developed additional computer solutions (using an IBM 704 digital computer) for the analysis of fixed end helicoidal girders subjected to uniform load over half span or subjected to a uniform torque per lineal foot of horizontal projection. In addition a computer program has been written which calculates and prints out the moments, torques, shears, and axial forces at any desired increment of horizontal angle once the redundants are known. Studies are also now underway to develop computer solutions for girders with other end conditions and for the general case of a helicoidal girder which forms a part of a three dimensional rigid frame system.

Attention will now be directed to the various comments of the discussers. The author has reviewed the publication by Menn[10] which was cited by Mr. Gedizli. This excellent piece of work includes the development of differential equations for the analyses of a: (1) ring girder; (2) helicoidal girder; (3) ring plate; and (4) helicoidal shell. Methods of solution are outlined but no tabulated numerical results are given. While the analysis of a helicoidal shell for certain special boundary conditions is feasible it is extremely complex mathematically. This fact is freely admitted by Menn in his conclusions where he also states that he believes in most cases the girder solution is generally adequate. The author has no personal knowledge of any helicoidal staircase which has been analyzed and designed using shell theory.

Mr. Gedizli and by personal correspondence Dr. Holdaway raise questions regarding assumed and actual fixity at the supports. Up to the present time insufficient studies have been made on the effect of the loss of fixity at a support to make any general statements. Where there is doubt as to the fixity, the author would recommend a procedure adopted by Mattock[11] and Morgan[12] in which they obtain an envelope of maximum moments for design from two separate solutions. The first solution is the same as the author's fully fixed end condition and the second is a solution based on simply supported ends together with sufficient assumptions to make the structure statically determinate for purposes of analysis.

Knowing the amount of computational labor involved, the author is grateful to Mr. Harboe for his careful check of the paper. With respect to the location of the section at which the maximum bending moments or torques occur no definite rule can be given, although in many cases the critical section is at the support. In a specific case

the moment and torque diagrams should be plotted using Eq. (12), (13), and (14). As mentioned earlier a computer program has recently been developed for this purpose.

As correctly pointed out by Mr. Harboe there is a typographical omission in the equation given for δ_{xw} on p. 1016. Each term on the right hand side of the equation should be multiplied by "sec α." In the actual computations for X_r and X_x it *was* included so these values are correct as tabulated. The fact that the effects of axial force and shear on the displacements have not been included should be apparent from the equation for δ_{xw} on p. 1016.

Through several personal communications Mr. Harboe and also Mr. C. A. Lee have suggested that for girders having high width—center line ratios b/R, an analysis due to a uniform torque per lineal foot of horizontal projection should be made and the internal forces produced should be added to those obtained for the uniform load case discussed in the paper. This torque loading results from the fact that a uniform vertical load such as dead load which is distributed over the width of the girder, has an eccentricity with respect to the center line of the girder. This eccentricity can be expressed by

$$e = \frac{b^2}{12R} \qquad (15)$$

From several cases he has checked the author has concluded that for b/R greater than 1/3 the contribution of this torque loading *may* be important and should be included in determining total internal forces and moments.

To facilitate the inclusion of the contribution of the torque loading, Table B gives results for X_r/R and X_x for a wide range of variables. These redundants, after being multiplied by the value of e in Eq. (15), may be added to those obtained for a vertical load of 1 lb per lineal ft of horizontal projection given in Table 1. Total moments due to both effects may then be found by:

$$M_r = m_{rw} + em_{rt} + X_x m_{rx} + X_r m_{rr} \qquad (16)$$

$$M_s = m_{sw} + em_{st} + X_x m_{sx} + X_r m_{sr} \qquad (17)$$

$$M_t = M_{tw} + em_{tt} + X_x m_{tx} + X_r m_{tr} \qquad (18)$$

Eq. (3) through (11) should be utilized in the above equations together with:

$$m_{rt} = -R\,(1 - \cos\theta) \qquad (19)$$

$$m_{st} = R \sin\theta \sin\alpha \qquad (20)$$

$$m_{tt} = R \sin\theta \cos\alpha \qquad (21)$$

m_{rt}, m_{st}, and m_{tt} represent bending and torsional moments in the girder due to a uniform torque loading of 1 ft-lb per lineal ft of horizontal projection with the redundants equal to zero. At every point along the longitudinal center line of the girder this distributed torque acts about a horizontal axis tangent to the center line.

TABLE B—REDUNDANTS AT MIDSPAN OF A HELICOIDAL GIRDER, FIXED AT THE ENDS, AND UNDER A UNIFORM TORQUE OF 1 FT-LB PER LINEAL FT OF HORIZONTAL PROJECTION

		$\alpha = 0$ deg		$\alpha = 10$ deg		$\alpha = 20$ deg		$\alpha = 30$ deg		$\alpha = 40$ deg	
ϕ, deg	b/h	X_r/R	X_s	X_r/R	X_s	X_r/R	X_s	X_r/R	X_s	X_r/R	X_s
45	4	—0.0461773	—	—0.0582804	—0.510506	—0.0801842	—0.754166	—0.0929676	—0.759314	—0.0920795	—0.652014
45	8	—0.0353261	—	—0.0718869	—1.24186	—0.106889	—1.27664	—0.111486	—0.994948	—0.0983644	—0.727412
45	16	—0.0279339	—	—0.106231	—2.49468	—0.128635	—1.68124	—0.119251	—1.11193	—0.0985328	—0.754490
60	4	—0.0906751	—	—0.121831	—0.742657	—0.165462	—0.944844	—0.180496	—0.845613	—0.169288	—0.674789
60	8	—0.0732117	—	—0.157928	—1.72500	—0.206115	—1.42991	—0.201271	—1.02230	—0.173064	—0.724448
60	16	—0.0611593	—	—0.206739	—2.87479	—0.225535	—1.68739	—0.204638	—1.08703	—0.169457	—0.737890
75	4	—0.156775	—	—0.216412	—0.990381	—0.278112	—1.07720	—0.287355	—0.880627	—0.261648	—0.673958
75	8	—0.133366	—	—0.274373	—2.06899	—0.320699	—1.46408	—0.303388	—1.00310	—0.260227	—0.706524
75	16	—0.116979	—	—0.326716	—2.97414	—0.332713	—1.62515	—0.301218	—1.04181	—0.252565	—0.714777
90	4	—0.247161	—	—0.337803	—1.21581	—0.404800	—1.14259	—0.402754	—0.877731	—0.362782	—0.658759
90	8	—0.220217	—	—0.405838	—2.25097	—0.439401	—1.43130	—0.410899	—0.962791	—0.355586	—0.682120
90	16	—0.201105	—	—0.448011	—2.91728	—0.443218	—1.53693	—0.403740	—0.989460	—0.344349	—0.689154
105	4	—0.360778	—	—0.474592	—1.38909	—0.533231	—1.15338	—0.519227	—0.851917	—0.468382	—0.635904
105	8	—0.333956	—	—0.538285	—2.30147	—0.555903	—1.36635	—0.519619	—0.914258	—0.456251	—0.655318
105	16	—0.314737	—	—0.566028	—2.78595	—0.553003	—1.44176	—0.508786	—0.935985	—0.442341	—0.663009
120	4	—0.491837	—	—0.613564	—1.49713	—0.656115	—1.12720	—0.633071	—0.813741	—0.576226	—0.609138
120	8	—0.468820	—	—0.663713	—2.26430	—0.667674	—1.28814	—0.627618	—0.863413	—0.560748	—0.627794
120	16	—0.452235	—	—0.678290	—2.62418	—0.660323	—1.34753	—0.614665	—0.883723	—0.545279	—0.637011
135	4	—0.630806	—	—0.744953	—1.54124	—0.770829	—1.07766	—0.743251	—0.768612	—0.685683	—0.579643
135	8	—0.613986	—	—0.779535	—2.17341	—0.774443	—1.20470	—0.734519	—0.811841	—0.668803	—0.599408
135	16	—0.601861	—	—0.784676	—2.45095	—0.764946	—1.25572	—0.721006	—0.832409	—0.653006	—0.610502
150	4	—0.767116	—	—0.864123	—1.52816	—0.877409	—1.01164	—0.850025	—0.717592	—0.796953	—0.546347
150	8	—0.757028	—	—0.886125	—2.04663	—0.876864	—1.11682	—0.840671	—0.757959	—0.780825	—0.568209
150	16	—0.749782	—	—0.885993	—2.26913	—0.867297	—1.16333	—0.828101	—0.779418	—0.766039	—0.581194
165	4	—0.892007	—	—0.970112	—1.46222	—0.976535	—0.929822	—0.953588	—0.658361	—0.909933	—0.506112
165	8	—0.887723	—	—0.984552	—1.88739	—0.975346	—1.02026	—0.946134	—0.697531	—0.896887	—0.530271
165	16	—0.884665	—	—0.982783	—2.07081	—0.967481	—1.06379	—0.935934	—0.719752	—0.884560	—0.544797
180	4	—1.00000	—	—1.06313	—1.34253	—1.06787	—0.828660	—1.05266	—0.586399	—1.02255	—0.454282
180	8	—1.00000	—	—1.07485	—1.68905	—1.06885	—0.907560	—1.04928	—0.624343	—1.01493	—0.479930
180	16	—1.00000	—	—1.07417	—1.84127	—1.06408	—0.947868	—1.04271	—0.646434	—1.00651	—0.495124

The detailed discussion by Mr. Bergman was most welcome as he and the author have long shared a mutual interest in helicoidal girders. Mr. Bergman cites tests run by Mr. Young and the author and states that "they carefully limited the range of validity of their findings to girders with a ratio of center line radius to width between 3 and infinity." Since only girders within this range were included in the experimental program[6] conclusions beyond this range would have been inappropriate.

In several instances Mr. Bergman expresses some qualms regarding the sensitivity of the internal forces in a helicoidal girder to possible errors in the assumed torsional stiffness of the girder. Table C indicates the results of a number of solutions carried out on the computer in which J_t was taken equal to 2, 1, and 0.5 times the value given by K_1bh^3. A study of the results indicates that this variation has a relatively small effect on the maximum values of M_r, M_s, and M_t. The greatest differences for the cases shown are 4 percent for M_r, 7 percent for M_s, and 14 percent for M_t.

Mr. Bergman makes an improper comparison when he compares the case of a deep curved cantilever bar loaded *in* its plane of major stiffness to a helicoidal cantilever which is essentially loaded *normal* to its plane of major stiffness. The deflection in either case can be easily calculated by adding to the virtual work expression for δ_{xw} on p. 1016 additional terms to account for the virtual work due to shear and axial force. In the curved bar loaded in its own plane this additional contribution is of the same order of magnitude as that due to bending about the x-axis and since moments about the r- and t-axes are zero for this case, shear and axial force effects should be included. For the heli-

TABLE C—EFFECT OF VARIATION IN ASSUMED VALUE OF J_t ON MAXIMUM VALUES OF M_r, M_s, AND M_t FOR GIRDER SUBJECTED TO 1 LB PER LINEAL FT OF HORIZONTAL PROJECTION

ϕ, deg	α, deg	b/h	$\frac{J_t}{K_1bh^3}$	Maximum M_r/wR^2	Maximum M_s/wR^2	Maximum M_t/wR^2
90	30	1	2.0	—0.770	—0.364	—0.122
90	30	1	1.0	—0.771	—0.372	—0.139
90	30	1	0.5	—0.767	—0.386	—0.154
90	30	16	2.0	—0.0812	—1.171	0.00596
90	30	16	1.0	—0.0815	—1.171	0.00572
90	30	16	0.5	—0.0820	—1.171	0.00540
135	30	1	2.0	—1.323	—0.796	—0.488
135	30	1	1.0	—1.292	—0.804	—0.472
135	30	1	0.5	—1.237	—0.819	—0.444
135	30	16	2.0	—0.420	—1.426	—0.0546
135	30	16	1.0	—0.424	—1.423	—0.0526
135	30	16	0.5	—0.431	—1.417	—0.0499
180	30	1	2.0	—1.987	—1.184	—1.085
180	30	1	1.0	—2.003	—1.240	—0.957
180	30	1	0.5	—1.983	—1.325	—0.829
180	30	16	2.0	—1.480	—1.796	—0.448
180	30	16	1.0	—1.520	—1.804	—0.401
180	30	16	0.5	—1.562	—2.138	—0.370

coidal cantilever of high b/R ratio the contribution of the moments about the r- and t-axes to the total virtual work is much greater than that due to moment about the x-axis or due to shear and axial force and the latter terms can be neglected without appreciable error. The author has computed the redundants in a number of cases with b/R ratios as high as 4/5 and found that the inclusion of the effect of shear and axial force gives a maximum difference of about only 2 percent from the values given in Table 1.

In conclusion the author once again wishes to thank those who participated in the discussion. Much work remains to be done before this fascinating type of structure is completely understood.

ADDITIONAL REFERENCES

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Discussion of a paper by E. Wolman:

Utility Poles of Reinforced and Prestressed Pipe*

By M. Z. COHN, A. SIEV, and AUTHOR

By M. Z. COHN†

In his interesting paper Mr. Wolman establishes design formulas for reinforced and prestressed pipe in which bending moments are the prevailing action.

Starting from the principles of the classical elastic theory, three cases of reinforcement distribution are considered: uniform distribution, local distribution on two opposite sides of the section, and cosine distribution.

The writer dealt with similar problems some years ago when engaged in research and design work on spun concrete poles for transmission lines. The need became apparent for a comprehensive investigation into the behavior of annular sections, not merely in the elastic, but in all loading stages. Such an approach to the problem was possible on the basis of Murashov's general theory[1] and has formed the subject of a paper by the writer.[2] He would like to add some comments resulting from his study on the analysis and construction of reinforced concrete annular members.

The discussion will be concerned with uniform reinforcement distribution alone, as it is the most consistent to the annular shape of reinforced concrete sections and is the most current in practice. Discussion will cover theoretical problems (including an alternative procedure of analysis based on ultimate strength theory and methods to estimate deflections and cracking at working loads) and practical design problems (limitations of steel ratio and distribution of bars).

ASSUMPTIONS

Notations used are those of the paper; additional notations are defined.

It is assumed that a section in bending passes through three distinct stages, each being characterized by strain distribution, steel and concrete stresses which can be approximated as shown in Fig. A.

Limit of the first stage is the appearance of cracks. Limit of the second stage is the formation of the plastic hinge (assumed to occur when steel is yielding up to the centroid of the tension zone.[3]) The third stage is reached at the ultimate strength of the section.

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†Research Engineer, Building Research Institute, Bucharest, Rumania.

Additional notation

d = distance between two bars in a cross section, in.
f_{ac} = average concrete limit strength in bending compression, psi
f_c' = cylinder strength of concrete, psi
f_t = limit strength of concrete in tension, psi
l_c = distance between cracks, in.
m = number of steel bars in a uniformly reinforced section
$n = \frac{E_s}{E_c}$, $n_1 = 2n$, $n_2 = 3n$—modular ratios of concrete
$q_1 = pn_1 = 2np$
$q_2 = pn_2 = 3np$
$q_s = f_y/f_{ac}$ = steel reinforcement parameter
u = ratio of total area and the total circumference of the steel reinforcement ($u = D/4$) when all bars have identical diameters D)
w = width of opened cracks, in.
B = EI (B_1, B_{2s}, B_{2c}) = flexural stiffness of beams in bending, psi
R = $f_c'/0.8$ = cubic strength of concrete, psi
S ($S_c = M/f_c$, $S_s = M/f_s$) = section modulus of reinforced concrete sections, cu in.
γ (γ_1, γ_2, γ_3) = neutral axis angle (See Fig. A)
λ_c = 0.6 = shape factor for annular sections
ψ = $\varepsilon_{as}/\varepsilon_s$ = ratio of average steel strain between cracks and strain of steel in the cracked section

Generally indices 1, 2, and 3 of various symbols indicate that they refer to the first, second, or third stages, respectively.

Subscripts s and c indicate that corresponding symbols refer to steel or concrete, respectively.

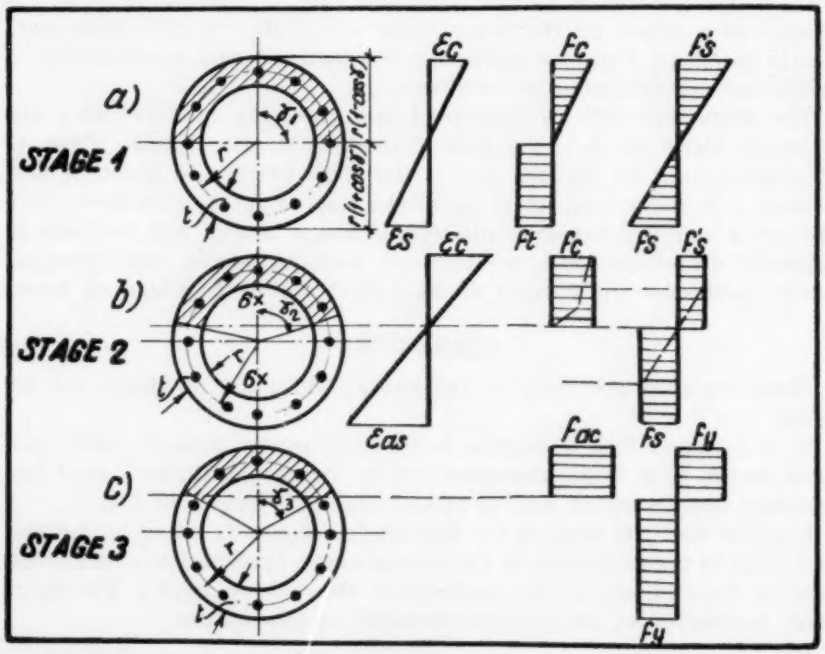


Fig. A—Strain and stress distributions in various loading stages of annular sections

Plastic properties of compressed concrete, as well as the influence of uncracked tensioned concrete between cracks, are taken into account by Murashov's theory.

For design it may be proved that adoption of distinct modular ratios for the first two loading stages are accurate enough. Values suggested in Reference 1 are $n_1 = 2n$ and $n_2 = 3n$ for the first and second stages, respectively ($n = E_s/E_c$).

Properties of materials, as given by Murashov, are listed in Table A. Conversion from metric units is made assuming cylinder strength to be 80 percent of the cubic strength for 20-cm square specimens.

Shape of the stress block has but a minor influence on moment values in Stages 2 and 3 and that is why a rectangular diagram is considered for the distribution of compressive stresses.

STRENGTH OF ANNULAR SECTIONS

For each loading stage two general equations may be written considering stress distributions of Fig. A:

$$\int f dA = 0 \qquad (15)$$

and

$$\int f y dA = M \qquad (16)$$

the integrals being extended over the active areas of steel and concrete proper to the stage considered.

Cracking moment

At the limit of the first stage, application of Eq. (15) yields the position of the neutral axis which in the final form is given by:

$$(\pi - \gamma_1)\,\frac{1 + \cos\gamma_1}{\sin\gamma_1 - \gamma_1\cos\gamma_1} = \frac{q_1 + 2}{q_1 + 1} \qquad (17)$$

Eq. (16) gives the cracking moment:

$$M_c = \eta_1 t r^2 f_t = S_1 f_t \qquad (18)$$

where

$$\eta_1 = 2(q_1 + 1)\left[\sin\gamma^2 + \frac{q_1 + 2}{q_1 + 1}\,\frac{2\gamma_1 - \sin 2\gamma_1}{4(1 + \cos\gamma_1)}\right] \qquad (19)$$

TABLE A — CONCRETE PROPERTIES

		Concrete strength, psi					
	f_c'	2000	2500	3000	3750	5000	6000
	$R = f_c'/0.8$	2500	3125	3750	4700	6250	7500
f_{cc}	psi	2200	2770	3270	3910	4980	5710
f_t	psi	215	265	295	340	405	455
$10^6 E_c$	psi	3.70	4.30	4.60	5.00	5.55	5.90
$n = E_s/E_c$	—	8.10	7.00	6.50	6.00	5.40	5.10

$E_s = 30 \times 10^6$ psi, $n_1 = 2n$, $n_2 = 3n$.

S_1 is the section modulus and f_t is the limit stress of concrete in tension (Table A).

It is easy to check, γ_1 is varying from 86 to 90 deg. Therefore $\gamma_1 = 90$ deg is a satisfactory approximation, and Eq. (19) becomes:

$$\eta_1 = 5.14 + 3.57\ q_1 \quad (20)$$

Moments in the second stage

For the working range Eq. (15) gives:

$$\frac{\pi - \gamma_2}{2} \cot^2 \frac{\gamma_2}{2} = 1 + \frac{1}{q_2} \quad (21)$$

from which γ_2 can be found, while Eq. (16) yields:

$$M = \eta_{2c} t r^2 f_c = S_c f_c \quad (22)$$

$$M = \eta_{2s} A_r r f_s = S_s f_s \quad (22.1)$$

in which

$$\eta_{2c} = 2\pi(1 + q_2) \frac{\sin \gamma_2}{\pi - \gamma_2} \quad (23)$$

and

$$\eta_{2s} = \frac{\sin \gamma_2}{\gamma_2} \quad (23.1)$$

S_c and S_s are the section moduli in the second stage

Ultimate moment

For the third stage, Eq. (15) and (16) give the angle of the compressed zone and the ultimate moment, respectively:

$$\gamma_3 = \frac{\pi q_3}{1 + 2q_3} \quad (24)$$

and

$$M_u = \eta_3 t r^2 f_{ac} \quad (25)$$

where

$$\eta_3 = 2 \sin \gamma_3 (1 + 2q_3) \quad (26)$$

f_{ac} is the average concrete limit stress in bending compression (Table A).

STIFFNESS OF ANNULAR SECTIONS

In the uncracked stage the stiffness $B_1 = (EI)_1$ can be expressed as:

$$B_1 = S_1 r (1 + \cos \gamma_1) \frac{E_c}{n_1} \quad (27)$$

Taking into account Eq. (18) and (20), and since $\gamma_1 \approx 90$ deg, the above equation becomes:

$$B_1 = \eta_1 t r^3 \frac{E_c}{2} = \beta_1 t r^3 E_c \quad (27.1)$$

with $\beta_1 = \eta_1/2$

In the second stage the stiffness may be expressed in terms of either the compression or the tension zone of the section. Curvature of the beam being constant under given bending moment M $(1/R = M/B_{2s} = M/B_{2c})$, it follows $B_{2s} = B_{2c}$ and choice of one or another value is matter of the designer's preference.

$$B_{2s} = \lambda_a \frac{E_s}{\psi} S_s r(1 + \cos \gamma_2) \quad \text{.....(28)}$$

$$B_{2c} = \lambda_a \frac{E_s}{n_2 \psi} S_c r(1 - \cos \gamma_2) \quad \text{.....(28.1)}$$

Here S_c and S_s are given by Eq. (22) and (22.1), λ_a is a modular shape factor which allows for the plastic behavior of the concrete in compression (for annular sections $\lambda_a = 0.6$ according to Reference 3) and ψ is a factor which takes into account the effect of uncracked tensioned concrete between the cracks.

$$B_{2s} = 0.6 \frac{E_s}{\psi} \eta_{2s} A_T r^2 (1 + \cos \gamma_2) = \beta_{2s} r^3 A_T \frac{E_s}{\psi} \quad \text{.....(29)}$$

$$B_{2c} = 0.6 \frac{E_s}{3n\psi} \eta_{2c} h r^3 (1 - \cos \gamma_2) = \beta_{2c} h r^3 \frac{E_s}{\psi} \quad \text{.....(29.1)}$$

where

$$\beta_{2s} = 0.6 \eta_{2s} (1 + \cos \gamma_2) \quad \text{.....(30)}$$

and

$$\beta_{2c} = \frac{0.2}{n} \eta_{2c} (1 - \cos \gamma_2) \quad \text{.....(30.1)}$$

For a given steel percentage p, and constant grade of concrete and steel strength β, it is obvious that the ψ factor has a great influence on the section's stiffness.

The factor ψ is defined as the ratio of the average steel strain between cracks (ε_{as}) and the strain of the cracked section (or free steel $-\ \varepsilon_s$) and hence $\psi \leq 1$.

It may be proved that its expression is:

$$\psi = \frac{\varepsilon_{as}}{\varepsilon_s} = 1 - \frac{n_1 f_t}{f_s} \left[\frac{2}{3} \phi - \frac{1 - \frac{n_1 f_t}{f_s} (\phi + 1)}{q_2} \right] \quad \text{.....(31)}$$

where

$$\phi = \frac{S_1}{n_1 S_s} - 1 = \frac{\eta_1}{2\pi q_1 \cdot \eta_{2s}} - 1 \quad \text{.....(32)}$$

A plot of ψ in terms of the steel parameter q_2 and the effective steel stress f_s is given in Fig. B

It is seen that ψ increases with the steel percentage and steel stresses approach unity for maximum values of p and f_s.

Analysis of moments and stiffnesses is greatly simplified by use of Table B, where coefficients γ_2, γ_3, η_{2s}, η_{2c}, η_3, β_1, β_{2s} and ϕ are tabulated in terms of the parameter q.

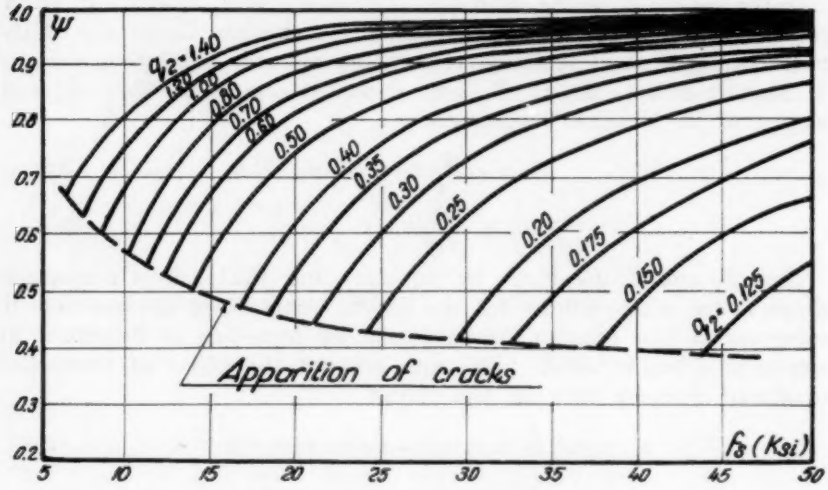


Fig. B—Variations of ψ factor

It should be noted that using Table B, q must be associated with the corresponding stage, i.e., for the first stage $q_1 = 2np$, for the second $q_2 = 3np$ and for the third $q_3 = pf_y/f_{ac}$.

Assuming γ_2 constant over the entire second stage, Table B allows the evaluation of working stresses in concrete and steel using Eq. (22) and (22.1):

$$f_c = M/S_c \quad \text{and} \quad f_s = M/S_s$$

CRACKING OF ANNULAR MEMBERS

The width of cracks, w, in members of annular section may be estimated as the elongation of the extreme fiber steel over the distance between two successive cracks, under the assumption of negligible elongation of tensioned concrete.

$$w = \varepsilon_{sa} l_c \qquad (33)$$

The average steel strain between two successive cracks is

$$\varepsilon_{sa} = \frac{f_s}{E_{sa}} = \frac{\psi f_s}{\lambda a E_s} = \frac{\psi f_s}{0.6\, E_s} \qquad (34)$$

According to Murashov's theory the distance between cracks is

$$l_c = n_1 u \phi \qquad (35)$$

Where u is the ratio of the total area and the total circumference of steel reinforcement ($u = D/4$ for all bars of identical diameter D), and ϕ given by Eq. (22) may be taken from Table B.

TABLE B — COEFFICIENTS FOR STRENGTH AND STIFFNESS ANALYSIS OF ANNULAR REINFORCED CONCRETE SECTIONS

q	Stage 1				Stage 2			Stage 3		q
(q_1, q_2, q_3)	η_1	β_1	γ_2	η_{2s}	η_{2c}	β_{2s}	ϕ	γ_3	η_3	(q_1, q_2, q_3)
0.05	5.32	2.66	0.780	0.905	2.00	0.930		0.143	0.32	0.05
0.10	5.50	2.75	0.888	0.872	2.40	0.854	13.75	0.262	0.62	0.10
0.15	5.68	2.84	0.980	0.847	3.75	0.791	9.35	0.362	0.92	0.15
0.20	5.85	2.92	1.040	0.827	3.10	0.747	7.13	0.449	1.22	0.20
0.25	6.03	3.01	1.093	0.812	3.40	0.710	5.76	0.523	1.50	0.25
0.30	6.21	3.10	1.137	0.796	3.70	0.682	4.85	0.589	1.78	0.30
0.35	6.39	3.19	1.168	0.786	3.95	0.657	4.20	0.646	2.05	0.35
0.40	6.57	3.28	1.197	0.777	4.20	0.637	3.69	0.698	2.32	0.40
0.45	6.75	3.37	1.220	0.769	4.45	0.620	3.31	0.744	2.58	0.45
0.50	6.93	3.46	1.240	0.762	4.70	0.606	2.97	0.785	2.83	0.50
0.55	7.11	3.55	1.260	0.756	4.93	0.592	2.76	0.823	3.08	0.55
0.60	7.28	3.64	1.274	0.750	5.16	0.581	2.49	0.856	3.33	0.60
0.65	7.46	3.73	1.290	0.745	5.38	0.570	2.34	0.187	3.57	0.65
0.70	7.64	3.82	1.305	0.740	5.60	0.560	2.19	0.916	3.81	0.70
0.75	7.82	3.91	1.317	0.735	5.82	0.551	2.04	0.942	4.05	0.75
0.80	8.00	4.00	1.328	0.730	6.04	0.543	1.89	0.965	4.28	0.80
0.85	8.18	4.09	1.337	0.727	6.25	0.537	1.77			
0.90	8.35	4.17	1.345	0.724	6.46	0.532	1.67	$M_c = \eta_1 tr^2 f_t$		
0.95	8.53	4.26	1.353	0.721	6.67	0.527	1.58			
1.00	8.71	4.35	1.360	0.718	6.89	0.522	1.50	$B_1 = \beta_1 tr^2 E_c$		
1.05	8.89	4.44	1.367	0.716	7.10	0.517	1.45			
1.10	9.07	4.53	1.375	0.714	7.31	0.512	1.40	$M = \eta_{2s} tr^2 f_s$		
1.15	9.25	4.62	1.382	0.712	7.52	0.507	1.32			
1.20	9.43	4.71	1.388	0.710	7.73	0.503	1.24	$M = \eta_{2c} A_T r f_s$		
1.25	9.61	4.80	1.394	0.708	7.94	0.500	1.19			
1.30	9.78	4.89	1.396	0.706	8.15	0.497	1.14	$B_{2s} = \beta_{2s} r^2 A_T E_s/\psi$		
1.35	9.96	4.98	1.398	0.704	8.35	0.495	1.10			
1.40	10.14	5.07	1.400	0.702	8.55	0.493	1.07	$M_u = \eta_3 tr^2 f_{sc}$		

Putting $\psi f_s \phi/0.6\, E_s = \lambda_c$ the crack width becomes finally

$$w = \frac{f_s}{0.6\, E_s}\, n_1 u \phi = u n_1 \lambda_c \qquad (36)$$

In Fig. C, λ_c is plotted against the steel stress f_s and steel parameter q_2.

EXAMPLE

Redesigning the pole of the example on p. 1051 of the paper, for $h = 30$ ft, $M = 240{,}000$ in.-lb, $r = 5.85$ in., and $t = 2$ in., crack width and deflection at working load are required.

The total reinforcement of the section was found, $A_T = 2.67$ sq in. and, therefore, $p = 2.67/2\pi \times 5.85 \times 2 = 3.63$ percent from elastic theory.

It will now be assumed that $K_s = 1.8$ and $f_y = 1.8 \times 24{,}000 = 43{,}000$ psi. For concrete, $K_c = 2.5$ and $f_c' = 2.5 \times 1200 = 3000$ psi and from Table A $f_{sc} = 3270$ psi.

Ultimate strength design

$$\eta_3 = \frac{M_u}{tr^2 f_{sc}} = \frac{1.8 \times 240{,}000}{2 \times 5.85^3 \times 3270} = 1.93$$

$$p = q_3 \frac{f_{sc}}{f_y} = \frac{0.328 \times 3270}{43{,}000} = 2.5 \text{ percent}$$

Table B gives $q_3 = 0.328$. It may be seen that ultimate strength design yields a 31 percent economy in reinforcement as compared to conventional theory.

$$A_T = 2\pi rtp = (2\pi \times 5.85^2)\ 2.5/100 = 1.84 \text{ sq in.} \quad \text{or } 9\ \#4 \text{ bars}$$

Cracking moment

$$q_1 = 2np = 2 \times 650 \times 2.5/100 = 0.325$$

Table B gives $\eta_1 = 6.30$

$$M_c = \eta_1 tr^2 f_t = 6.30 \times 2 \times 5.85^2 \times 293 = 127{,}000 \text{ in.-lb}$$

Width of cracks at working load

$$q_2 = 3np = 3 \times 6.50 \times 2.5/100 = 0.488$$

Table B gives $\eta_{2s} = 0.764$ and $\beta_{2s} = 0.609$

$$f_s = \frac{M}{\eta_{2s} A_T r} = \frac{240{,}000}{0.764 \times 1.84 \times 5.85} = 29{,}200 \text{ psi}$$

with $q_2 = 0.488$ and $f_s = 29.2$ ksi, $\psi = 0.88$ is obtained from Fig. B and $\lambda_c = 4.3 \times 10^{-3}$ from Fig. C.
Hence $w = un_1\lambda_c = D/4\, n_1 \lambda_c = \frac{1}{8} \times 2 \times 6.5 \times 4.3/1000 = 0.007$ in.

Deflection at working load

$$B_{2s} = \beta_{2s} r^3 A_T \frac{E_s}{\psi} = 0.609 \times 5.85^2 \times 1.84 \times \frac{30 \times 10^6}{0.88} = 1.32 \times 10^9$$

$$\delta = \frac{Mh^2}{3B} = \frac{240{,}000 \times 12^2 \times 30^2}{3 \times 1.32 \times 10^9} = 7.85 \text{ in.}$$

LIMITATIONS OF STEEL REINFORCEMENT

Eq. (25) is valid only if steel yielding begins prior or simultaneously to concrete crushing. According to the Rumanian[4] and Soviet[5] specifications for reinforced concrete design this condition is fulfilled when

$$q_3 \leq 0.8 \qquad (37)$$

Therefrom results the balanced percentage of reinforcement for annular reinforced concrete sections, ($p_o = 0.8\, f_{ac}/f_y$), which is tabulated for various steel and concrete qualities in Table C.

A theoretical lower limit of the steel percentage may be derived from the condition that cracking moment, Eq. (18), to be at least equal to the moment of the same unreinforced section. Figures thus obtained being low, it seems more reasonable to derive it on practical considerations as follows.

If d is the distance between m steel bars of a section, the circumference will be $2\pi r = md$ and the total steel area $A_T = 2\pi rtp = mdtp$. It results that

$$d = \frac{A_T}{mpt} \qquad (38)$$

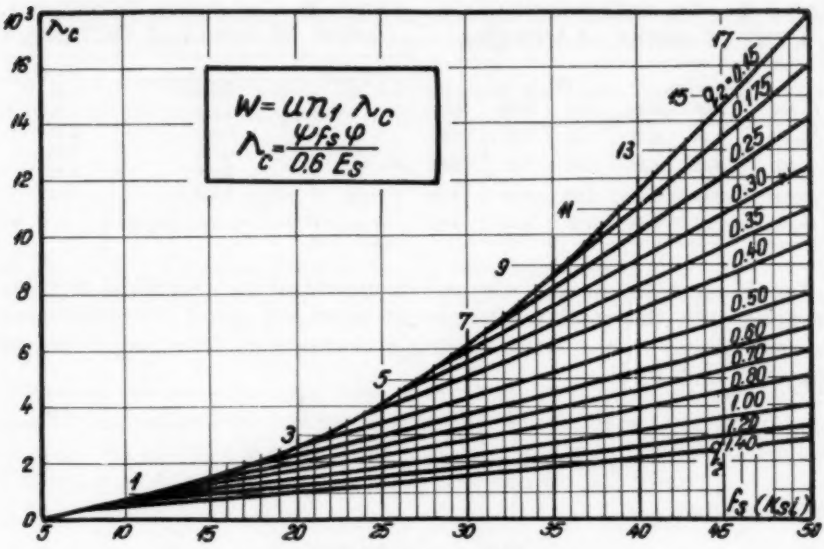


Fig. C—Variations of λ_c factor

Let d_o be the maximum permissible distance (from design and construction considerations). Hence

$$\frac{A_T}{mpt} \leq d_o \quad \text{and} \quad p \geq \frac{A_T}{d_o mt} \qquad (39)$$

If all bars are of the same diameter D, $A_T/m = \pi D^2/4$ and Eq. (39) becomes:

$$p \geq \frac{\pi D^2}{4\, d_o t} \qquad (39.1)$$

A minimum value of p will be obtained when t and d_o are the greatest possible. Practically, assuming for annular poles $d_{o\,max} = 6$ in. and $t_{max} = 3$ in., for various bar diameters the values of Table D result.

Table D shows steel ratios of annular sections to be generally high. Strength and economical considerations seem to indicate a variation of p from 2 to 4 percent to be reasonable in design practice.

REINFORCEMENT ARRANGEMENT

In deriving formulas for M in both the original paper and the above discussion the assumption was made that reinforcement is located on the medial circumference of the concrete area.

As the author points out it would be more logical to arrange steel at the most extreme fiber to obtain the best use of its carrying capacity. Thus, it may be desirable to know the amount of error involved in the assumption used.

TABLE C — BALANCED REINFORCEMENT OF ANNULAR SECTIONS

f_y, psi	f_c' psi				
	2000	2500	3000	3750	5000
35,000	5.04	6.35	8.58	8.94	11.40
40,000	4.42	5.55	7.50	7.80	9.95
45,000	3.92	4.94	6.68	6.94	8.84
50,000	3.53	4.44	6.00	6.25	7.95

TABLE D — MINIMUM REINFORCEMENT OF ANNULAR SECTIONS

Bar No.	D, in.	p_{min}
4	0.500	1.11
5	0.625	1.72
6	0.750	2.45
7	0.875	3.34
8	1.000	4.40
9	1.128	5.55

$p \geqq \pi D^2/4d_s t$ in percent, $d_{s\,max} = 6$ in., $t_{max} = 3$ in.

Let M_u and M_u' be the ultimate moments of two identical annular sections with the steel reinforcement arranged along circumferences of radii r and r', the latter having a covering c. Then, according to Eq. (25) and (26):

$$M_u = 2tr^2 f_{sc} \sin \gamma_s (1 + 2q_s) \quad (40)$$

$$M_u' = 2tr^2 f_{sc} \sin \gamma_s \left(1 + 2q_s \frac{r'}{r}\right) \quad (40.1)$$

It follows that

$$\frac{M_u'}{M_u} = \frac{1 + 2q_s r'/r}{1 + 2q_s} \quad (41)$$

The ratio will be a maximum with maximum values of r'/r and $q_{s\,max} = 0.8$

$$r' = r + \frac{t}{2} - c - \frac{D}{2} \quad (42)$$

From design experience $r_{max} = 10$ in., $t_{max} = 3$ in., $c_{min} = 3/4$ in. and $D_{min} = 1/2$ in. $r' = 10.5$ in. is obtained, and therefore $(r'/r)_{max} = 1.05$

With the above figures, the maximum value M_u'/M_u results from Eq. (41):

$$\left(\frac{M_u'}{M_u}\right)_{max} = \frac{1 + 2 \times 0.8 \times 1.05}{1 + 2 \times 0.8} = 1.03$$

Hence, assumption of steel arrangement on the medial circumference of reinforced concrete results in conservative ultimate moment values, the error being (for current cases) less than 3 percent on the safe side.

REFERENCES

1. Murashov, V. I., "Cracking Resistance, Stiffness, and Strength of Reinforced Concrete," *Mashstrojizdat* (Moscow), 1950 (in Russian).

2. Cohn, M., "Analysis of Reinforced Concrete Annular Sections Subjected to Bending," *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 6, No. 8-9, Aug.-Sept. 1955, pp. 452-460. (in Rumanian)

3. Murashov, V. I., "A New Method for the Analysis of Reinforced Concrete Chimneys," *Stroitelnaja Promishlennosti* (Moscow), No. 6, June 1951. (in Russian)

4. STAS 1546-50. "Analysis of Reinforced Concrete Sections According to the Ultimate Strength Theory" (Rumanian specificiations), Bucharest, 1950. (in Rumanian)

5. N i TU 123-55, "Provisions for the Design of Plain and Reinforced Concrete Members," (Russian specifications), Moscow, 1955. (in Russian)

By A. SIEV*

The writer feels unable to agree with the author with regard to two of his conclusions.

A. Reasons for cosine distribution of reinforcement.

The author mentions the "exact" theory as source of the equation

$$n_s = \frac{M}{\pi r^2} \cos \phi$$

However, the same formula may be obtained by assuming a homogeneous uncracked section (concrete being capable of tensile stresses, contrary to the author's assumptions) and using the author's own formulas. Putting $x = 0$; $a = 0$; $l = 2\xi$ in Eq. (1) and Eq. (5) then

$$M = P_s l = \pi r^2 t f_s$$

Putting $n_x = t f_c$, transposing and noting that

$$n_s = n_{s\,max} \frac{y}{x} = n_{s\,max} \cos \phi$$

we arrive at the above formula. The fact that the "exact" theory yields the same result as the elementary theory of strength of materials is explained by the behavior of the pole as a long shell. *This reasoning shows that the strains in the fibers of the pole are proportional to their distance from the neutral axis, and so are the stresses, and a cosine distribution of steel does not produce "equal stress at any section" as claimed by the author.*

The use of a cosine distribution may be accepted, though as an arbitrary intermediate stage between reinforcement equally distributed around the circumference and that of reinforcement concentrated at opposite points, with the advantage of ability to withstand bending moments in the perpendicular direction, as mentioned by the author. Derivation of formulas must be corrected accordingly.

B. Material economy of prestressed concrete.

The third statement in the summary, namely, that prestressing does not offer as much material advantage over simple reinforcement as in cases of structures subjected to loads from one direction, may be misleading. This is related to the author's error in stating that "the reinforcement is strained to the limit $2 f_s$." It is a known fact that stresses in prestressed steel do not vary much during loading and unloading of the beams due to the combined action of steel and concrete.

*Member American Concrete Institute, Lecturer, Technion-Israel Institute of Technology, Haifa, Israel.

TABLE E—COMPARISON OF ORDINARY REINFORCED AND PRESTRESSED CONCRETE

M = 240,000 in.-lb

	p_s, psi	f_c, psi	n	r, in.	t, in.	D, in.	A_T sq in.	A_T per-cent	A_c sq in.	A_c per-cent
Example 1 author	24,000	1200	10	5.85	2	13.7	2.67	100	73	100
Example 2 concentrated reinforcement	24,000	1200	10	4.40	2	10.8	2.43	91	55.5	76
Example 3 prestress	72,250	2000	4.75	4.40	2	10.8	1.47	55	55.5	76

Let us calculate the permissible prestress in the reinforcement with a view to obtaining the permissible stress in bending: f_s, prestress, $+ nf_c = 2/3 \times$ ultimate steel stress. Now, for the concrete used E_c, according to Ros,[6] is 5.9×10^6 psi, which yields $n = 4.75$.

Therefore:

$$f_s = \frac{2}{3} \times 120{,}000 - 4.75 \times 1000 = 80{,}000 - 4750 = 75{,}250 \text{ psi.}$$

This number is almost double that used by the author (40,000 psi). With the aid of a simple "trick," namely locating the steel symmetrically on (or near) the neutral axis it can be seen that there is no change in strain in bending and the full prestress of $f_s = 80{,}000$ psi may be used.

Using the author's data and formulas, the pole in the author's example has been calculated for the case of reinforcement concentrated at opposite points and for prestress ($t = 2$ in.). Results are given in Table E. The prestressed pole is more economical in steel, and economy in concrete may be achieved by using higher quality concrete, (which is easily produced under factory conditions). In this case we have a pole which seems more slender externally. This seems to the writer to be an important advantage from the architectural point of view.

REFERENCE

6. Komendant, August E., *Prestressed Concrete Structures*, McGraw-Hill Book Co., Inc., New York, 1952, p. 37.

AUTHOR'S CLOSURE

Mr. Siev is to be commended for deriving a part of the formulas of the section "cosine distribution of reinforcement" by the theory of strength of materials. But the more correct his statement that the stresses are proportional to the distances from the neutral axis, the less is understood his conclusion that the cosine distribution of steel does not ensure equal stress at any section.

If the conclusion arrived at by Mr. Siev is based on a straight line proportionality then he is mistaken, as the distances are from the

circumference of a circle and are in fact the cosines. If Mr. Siev has some other thought in view, then it is regretted that he is not more explicit.

Comparing the values $n_x = M/(\pi r^2) \times \cos \phi$ and $n_{x\,max} = M\pi r^2$ in the paper, the relation $n_x = n_{x\,max} \times \cos \phi$ is obvious and, as the author has the same cosine stress relation as deduced by Mr. Siev, and as the reinforcement is distributed to ensure equal stress at any section, the objection raised by Mr. Siev cannot be accepted. Also, the author cannot agree to Mr. Siev's assumption of "homogeneous uncracked concrete capable of withstanding tensile stresses" as the main advantage of longevity of concrete poles over steel and timber pylons will immediately be endangered by rusting of reinforcement due to cracks developed in the concrete subjected to calculated tension. As even minute cracks have the tendency to enlarge due to frost and other factors Mr. Siev's assumption cannot be accepted for practical reasons. If uncracked concrete is to be considered then certainly not concrete designed for tension.

Mr. Siev's statement that stresses in prestressed steel do not vary much during loading and unloading of the beams is correct only in regard to long-span structures with "nonbonded prestressed reinforcement" such as introduced by Dischinger and Finsterwalder for trussed beams and arches. In these structures the tensioning is effected by dead load and preloading simultaneously with the removal of the scaffolding.

The utility poles cannot, however, be manufactured according to this principle and, most obviously, they belong to the sort of structures with "bonded prestressed reinforcement" in which, to use Mr. Siev's references,* the stresses in steel due to prestressing and the stresses in steel due to bending moments are caused separately and should be added together. If the strain ability of steel left after prestressing is insufficient, the reinforcement will break.† Thus, Mr. Siev's reasoning does not apply to the prestressing of utility poles.

Mr. Cohn's discussion is a valuable paper in itself, using the ultimate strength theory for the problem of the original paper. The objection to this procedure is based on the following considerations:

(1) As the poles are subjected to impact such as snapping of wires and sudden gusts of wind, the question is whether plastic hinges will develop instantly. To the author's knowledge bridges and water tanks are not permitted to be designed according to the ultimate strength theory, both being considered as subjected to impact.

(2) The assumption of cracks by this theory cannot be agreed upon as explained above in the answer given to Mr. Siev.

The author wishes to thank Mr. Siev and especially Mr. Cohn for their discussion. Although the author does not agree with the latter he sees in Mr. Cohn's discussion a rational extension of his own paper.

*Reference 6, pp. 55-56.
†Reference 6, p. 68.

Discussion of a paper by Chester P. Siess:

Research, Building Codes, and Engineering Practice*

By EDWARD A. ABDUN-NUR, GEOFFREY BROCK, A. CÖUARD, ANTHONY HOADLEY, ERNEST KALVE, and AUTHOR

By EDWARD A. ABDUN-NUR†

Professor Siess has given the profession an interesting and thought-provoking paper, and has made an excellent case for codes. The writer wishes to take this opportunity to congratulate him on this fine piece of work.

Like everything else in life, even though codes have an important place in engineering practice, they do have some weaknesses, limitations, and disadvantages. The writer agrees with the author in the general over-all picture, but feels that the "other side," which the author has not brought out, should receive an airing at the same time, so as to present to the reader a more realistic picture than one side alone can do. This discussion will attempt to bring out this other side.

The diagram given in Fig. 1 is most interesting, but implies that the practicing engineer is largely a walking encyclopedia of knowledge. It fails to take into account that the engineer is primarily an individual skilled in taking knowledge, and through thought processes, experience, judgment, and insight develops solutions to problems that are presented to him. It is the last two factors of judgment and "insight" or "feel of engineering" that make the difference between a professional man and a technician. The author's diagram is a perfectly fitting representation of a technician but certainly not of a professional engineer.

Rarely does a problem present itself to the practicing engineer that does not require research on his part to develop realistic solutions. This research may not be formalized. It may be a simple experiment in the field, or in the laboratory or in the kitchen sink; or it may be research conducted at the engineer's desk or in the library. It all adds up to the same thing — an extension or rearrangement of formalized and recorded knowledge. For this reason, the writer suggests that the line from Research to Practice should be made at least as heavy as that from Codes to Practice and the importance of the thinking processes, judgment, and insight in the solution of problems in practice should be acknowledged by a heavier and bigger feed-back loop in the Practice circle.

*ACI JOURNAL, V. 31, No. 11, May 1960 (*Proceedings* V. 56), p. 1105. Disc. 56-55 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 6, Part 2, Dec. 1960 (*Proceedings* V. 56).

†Member American Concrete Institute, Consulting Engineer, Denver, Colo.

The author implies that it is not possible, practical, or safe for an engineer to utilize the results of research, until it has been digested for him and formalized by a code-writing body. Again, if the intention is to describe a technician's possible skills, the author may be perfectly correct, but it denies the very nature of professionalism to claim that the professional engineer is incapable of sufficient judgment to do these things for himself.

This much for the philosophical approach. Now to the limitations and disadvantages of codes per se. In the first place, inasmuch as codes are printed documents, they become static — at least for a period — while practice and knowledge keep advancing. For example, the ACI Building Code is dated 1956 and a revision is not expected until 1962. Therefore, right now, it is 4 years behind time and will lag 6 years by the time a revision is published.

Secondly, a code is even behind the times at the moment it is published. This is so because it is the result of committee action. Everyone who has worked on committees knows that there are bound to be differences of opinion among committee members, and the larger the committee the more the differences. To resolve such differences and get something out of committee, compromises are made, which result in watering down of up-to-date knowledge and information. In this connection, William H. Whyte, Jr., in his book, *The Organization Man* sums up the limitations of committee action in the following apt words:

> The most misguided attempt at false collectivization is the current attempt to see the group as a creative vehicle. Can it be? People very rarely *think* in groups, they talk together, they exchange information, they adjudiciate, they reach compromises. But they do not think; they do not create. .

The result of such compromises is not only delay in the application of available knowledge, but also in a conservatism that is evidenced in excessive factors of safety. The writer does not wish to imply that committee work can be done away with. On the contrary, committees are needed and have a definite place in our social structure. But one must face it — they have their limitations, which are at times serious.

Granted—conservatism is needed in a code that is going to be applied by every technician and even by persons without technical background. But no progressive practicing engineer can afford to be as much behind available knowledge as codes are, nor can he afford to penalize his clients as a result of that and load them with the added cost.

Some working procedure to permit the engineer who has the skills, ability, judgment and insight to make use of available knowledge with safety, should be incorporated into every code to minimize its static nature and the conservatism inherent in such documents.

By GEOFFREY BROCK*

This interesting paper concerns a subject which deserves more attention. Generally, Professor Siess has sought to justify the present status of codes, writing that his paper should not be interpreted as any criticism of the ACI code. In contrast this discussion will be restricted to a presentation of some criticisms of the philosophy of mandatory codes.

Codes of practice or regulations for the use of reinforced concrete were generally introduced some 50 years ago when too little was known about it. Before that the pioneers had built many successful structures and it would be difficult to show that the introduction of codes raised the general standard of design. Indeed, the replacement of many independent conceptions, each perhaps containing a little truth, by a unified code, which we are now rejecting as faulty, could well be described as a mistake.

Professor Siess has written "In the proper hands, our codes lead to good structures," but surely good structures can always be expected from the proper hands. The tacit assumption underlying codes seems rather that good structures can be designed by anyone who faithfully adheres to a set of rules. But in practice this often means that even the imaginative and technically competent designer must be bound by regulations designed to be "a check on undiscriminating unintelligence."[1]

The internationally formulated EUSEC definition of a professional engineer includes the sentence "His education will have been such as to make him capable of closely and continuously following progress in his branch of engineering science by consulting newly-published work on a world-wide basis, assimilating such information and applying it independently." While the author acknowledges that many engineers can do this, he goes on to state that individual study "might well be replaced by an alternate procedure in which a small group of engineers is formed into a committee to study collectively the results of researches and current practices." He does not give any reason for preferring the alternative, which is difficult to reconcile with professional independence. Perhaps the codes have, through their restriction of individual thought and experiment in design been responsible for the infrequent breakthroughs in knowledge to which the paper refers.

A major difficulty facing the code writer is that pronouncements must be made on matters which are not currently fully understood. However, codes seldom make it clear that some of their provisions are less justified than others: they are all given the same force of law. How much more useful they would be if they cited evidence for each provision and the reasoning behind any simplification involved, such as that mentioned by the author in deriving the 1940 Joint Committee Report from the 1926 Westergaard paper.

*Member American Concrete Institute, Lecturer in Civil Engineering, University of Birmingham, England.

This reluctance to justify rules has also spread to the textbooks. Early writers, such as Turneaure and Maurer or Taylor and Thompson, devoted chapters to reporting tests. Many contemporary writers prefer to report the codes instead. The codes and the texts conspire to make it difficult for the student to determine which rules are based on experiment and which on "informed opinion."

A lack of candor in drafting rules can lead to the perpetuation of unjustified methods and the suppression of the truth. Professor Siess has drawn attention to the early work of Talbot on shear in beams which was ignored for 30 years. The reason is not hard to find; the codes by-passed the questions raised by this work and no one was interested. Perhaps our present state of knowledge would now be sounder had the early code writers not blindly accepted Morsch's treatment of shearing stresses.

The division of the source of knowledge into research and experience seems arbitrary. Knowledge is always derived from experience, and useful research is merely experience gained under controlled conditions. The experience to be gained of the behavior and serviceability of concrete structures from casual observation during normal practice can be over-rated. Firstly, designers often do not construct and constructors may not design, so that the effects of design on construction and performance are not readily noted. Secondly, observation is necessary for many years after construction, and those interested may no longer be available. For example, it might be desired to study the long-term cracking of shell roofs. But the outsides of these structures are covered with waterproofing material, the insides are disguised or inaccessible, and neither the designer nor the constructor may care to approach the unsuspecting owner for permission to examine the cracks in his roof.

It is unfortunate that in British practice the responsibility for codes has fallen to the British Standards Institution, a body originally formed for the praiseworthy purpose of standardizing the specifications of manufacturing components, but it does serve as a reminder of the present acceptance of the code. In the current British code no provision is made for considering the results of tests to destruction on models or full-size replicas in lieu of conventional calculations. The writer was recently informed that under the present Australian code model tests were acceptable only if made on elastic models. Thus one would be permitted to base designs of a reinforced concrete structure on the study of a perspex model but not on one of reinforced concrete.

These inconsistencies lead to the conclusion that codes are not the unmixed blessing which is sometimes supposed, tending to bring all design to a uniform level of mediocrity. They could be improved by freeing designers of any compulsion to use them, and transforming restrictive sets of rules into real guides to true understanding. If code

writers assume the relationship of teacher to the professional engineer, they should accept the discipline of marshaling the evidence and justifying the provisions in the document itself. How much more would we learn from the early codes had this been done.

Finally, engineers should be much more alert to criticize codes. Freyssinet has written "Some people will say that a respect for regulations is essential and that engineers need not check the hypotheses on which they are based. It is a convenient theory, but a false one. Men who draw up regulations can be wrong like other men."[13]

REFERENCE

13. Freyssinet, E., "The Birth of Prestressing," Translated from *Travaux* (Paris), *Library translation* No. 59, Cement and Concrete Association, London, 1956.

By A. CÖUARD*

Professor Siess's paper is one of the most stimulating I have read on the subject. However, it seems to me, in the light of Continental experience, that codes as described by the author can be extremely dangerous and conducive to paralysis of progress.

Reinforced concrete and prestressed concrete were launched through free enterprise and daring, with the sanction of the personal responsibility of the engineers and contractors who offered their clients structures whose serviceability under specified loads was the only contract obligation.

When it was seen that concrete was successful and that there was money in it, a lot of people went into the field who had no business to tackle it because they did not have the proper qualifications, and there were a lot of accidents. Price competition played its role too, the net result being that a kind of censorship was felt needed to prevent excesses. Codes of practice were introduced to put limits on the daring of the entrepreneurs first, and to try to fix the boundaries of responsibility between design and execution, as litigations in court could not be judged without reference to impersonal rules.

The trouble was that such rules could not be valid if the theories on which they were based were wrong, and experience was stifled by those very rules. Unhappily, as I think I have proved with the help of the experiments I have cited in other publications, the elastic theory was false when applied to solids whose angle of internal friction is not zero, and concrete is susceptible to creep.

We must face facts, however, and recognize that the reasons why the serviceability of a structure is inadequate are, by order of importance: settlement of the foundations, shear ruptures, and excessive deflections.

Anyone who would dare to propose a code of practice on soil me-

*Member American Concrete Institute, Consulting Engineer, Paris, France.

chanics would, for the time being, be simply laughed at. Soil mechanics is still an art and not a science.

As for shear, one has only to read all the interesting papers published in the ACI JOURNAL to be convinced that a lot of study and experimenting has to be done before we can pretend to what could be called knowledge.

As far as deflections are concerned, if the problem is theoretically solved, it is not at all the same practically through the interference of creep on which a lot of study has to be done, as nobody knows how to calculate I and E in such a way that the conclusions will be valid in 2 or 3 years, depending on the loads effectively applied.

On top of all those theoretical voids in our knowledge, there are the discrepancies between what is assumed for the mechanical characteristics of the concrete, and what is obtained in fact in actual structures: it seems that until now, thought has been fixed on the compressive strength mainly, without enough consideration being given to tensile strength of the same concrete, which is much more important, especially for shear strength.

If we want to elucidate those problems, so as to be able to reduce the actual safety coefficients which are so high that most calculations are really more "eye-wash" than we would like to admit, we then have only two ways of doing it: by experience through the trade, but the very existence of a compulsory code of practice forbids it, or if it is done surreptitiously, it cannot be invoked publicly; or by research, but then we fall on the dangers inherent to the procedure envisaged by the author (p. 1107) in referring to a "small group of engineers formed into a committee..."

It seems too much to hope from human nature, and even if this first committee was composed of angels, without any pride in even their own pet theories or previously publicized opinions, it would not be long before those positions would be monopolized by a clique of some sort, motivated by less altruistic motives: it happened elsewhere...

And the danger would be magnified if an international code of practice was conceived, because one can at least hope for some competition in a divided world, which gives truth and progress its chance, however slight as is still the case, the elastic theory notwithstanding.

So, in the writer's opinion, the only road to progress would be to have a code of practice which would be more exacting for the control of the concrete as used on the site and on the conformity of the reinforcement with the plans, which should be the responsibility of the contractor; the signing of the plans by the consulting or design engineer so that his personal responsibility would be engaged, with the obligation for him to be insured, as a driver is. However, insurance companies should be restrained so that their technical checkers would not become

too powerful, with almost dictatorial powers leaving them sole judgment on final design, as has sometimes been the experience on the Continent. The contractors, or the design engineer, who would have accidents would be quickly eliminated from any responsible position through the higher premiums, or revoking of their contracts by the insurance companies. In that way, the design code of practice could be only on a recommendation basis, which could be invoked by the courts in case of litigation based on an unserviceable structure, but could not be invoked if the structure is serviceable according to the contract. So only the competent would dare go into an industry which should do all it can to attract the best brains instead of putting them in a restricting straight-jacket: after all no progress can be hoped from a Procrustean bed whose only use is to cut off the emerging head.

To use a code of practice as an abridged night course to put any one on an equal basis with the best in the profession may be a way to proletarize the engineering profession, but not to progress.

By ANTHONY HOADLEY*

In addition to presenting an excellent statement of the nature and philosophy of building codes in general, Professor Siess has given us a valuable review of the development of the ACI Building Code. This review is of particular interest to the student in that it gives an indication of the reasoning behind various changes which have taken place in the Code and also gives an indication of the practical experience, experimental work or theoretical studies on which the various requirements are based.

Professor Siess points out that the Code requirements for the design of a column footing are based on the use of 85 percent of the moment at the face of the column carried by the footing, this requirement being based on tests made by Talbot half a century ago. Tests made by Richart in 1946 on reinforced footings show that the footing should be designed on the basis of the full cantilever moment at the face of the column. The writer has not seen any technical discussion challenging the correctness of Professor Richart's conclusions as to the use of a 100 percent moment factor for square reinforced footings.

The writer heartily supports Professor Siess' plea that this Code requirement be revised in light of our present knowledge. The argument that column footings designed on the 85 percent rule have not failed does not greatly impress the writer. Most footings are buried from sight long before the load they carry would cause any signs of distress to say nothing of failure. If the 85 percent rule to be followed for moment, the writer would be just as happy to use 85 percent of the computed shear in determining the diagonal tension stress. He

*Member American Concrete Institute, Professor, Civil Engineering Department, Union College, Schenectady, N.Y.

would at least have a balanced design, which is more than results from following the ACI Building Code.

The writer is pleased to see that two recent textbooks on reinforced concrete design, one by Phil M. Ferguson, past-president of ACI, and the other by George E. Large, both discuss the Richart tests and advocate the use of the full cantilever moment for determining the bending stresses in the design of column footings.

By ERNEST KALVE*

While I have no intention to criticize or minimize the importance of the author's paper, I would like to discuss this subject from a different point of view.

The author says: "We see then that building codes are, or can be, a powerful and effective means of collecting and disseminating knowledge." I think that building codes should set only the boundaries of safety around all aspects of its subject to prevent politicians, venturesome or unskilled, from trespassing the dangerous territory, but leave the engineering to the engineers. The code should not be a "powerful" collection of engineering knowledge and should not limit engineers to tried and tested methods only, but should have provisions for any safe progress and new developments. I am sure that this would encourage many engineers to assume the responsibility for great and advanced designs.

Our present building codes are overfilled with outdated engineering practices and with powerful and effective restrictions for new and progressive designs. Such limitations are unduly curbing the initiative and progress of civil engineering and also are lowering the quality and prestige of engineers.

The educational program of civil engineering is undergoing fundamental changes to improve the quality of our future engineers. But, without fundamental changes in our building codes, the educator's effort will be worthless.

AUTHOR'S CLOSURE

Mr. Cöuard's philosophical and sometimes very realistic comments regarding the undesirable features of standardized codes are very lucidly, and even strongly, presented. It seems clear from his remarks that he falls within the category the writer had in mind when he stated on p. 1121 that "Many engineers look upon codes as a necessary, or even an unnecessary evil, as something that prevents them from doing what they want to do and what they know is right and know is best." Nevertheless, as Mr. Cöuard himself so ably points out, there are many gaps in the knowledge of even the best engineers. He men-

*Member American Concrete Institute, Structural Designer, Schmidt, Garden & Erikson, Architects and Engineers, Chicago, Ill.

tions particularly shear and deflection. If these gaps exist, what then is the practicing engineer to do when it becomes necessary to consider such matters in the design of structures? Although committees are notoriously inefficient and excessively slow, they still represent one measure whereby conservative rules for design can be set down and the blame for any inadequacies due to the lack of knowledge can be placed not on the individual but on the profession where it belongs. His proposal that designers be given much more freedom and that those who are incompetent be eliminated as thier structures fail or become unserviceable is not consistent with the current philosophy regarding building codes in this country. In the United States at least, a building code is a legal document, adopted by the legislative body of a city or state, and intended primarily to protect the public health and safety. It is for this reason that codes most frequently are concerned more with safety than with serviceability. The writer doubts whether the ACI Code in itself is sufficient to guarantee the serviceability of any structure designed in accordance with it. Nevertheless, the writer must agree with Mr. Cöuard that it would be highly desirable if all engineers could attain that standard of professional competence which would permit them to work without the restrictions of a formalized code. This, however, must be considered, at least in the United States, as a goal toward which the engineering profession and the engineering educators should strive.

The writer is grateful to Mr. Abdun-Nur for presenting the "other side." It was not the writer's intention to make a "case" for codes. The writer knows, probably better than most, the weaknesses and limitations of codes, and some of these were pointed out in the paper. It must be admitted, however, that emphasis was placed on the beneficial aspects of codes, especially as a means for translating the results of research into something that can be used in practice.

Mr. Abdun-Nur is undoubtedly one of the "many engineers who have the time, skill, and patience to make detailed studies of research reports and technical papers, to dig out the significant bits and pieces of knowledge, and to apply the results directly in their engineering practice." The writer believes, however, that it is a mistake to place all professional engineers in this category and to relegate all without these qualifications to the category of technicians. There are undoubtedly many competent practicing engineers who have the ability but who are not able to find the time to keep up with all of the research bearing on their day to day problems and to digest this information and utilize it in their practice. These men should not be called technicians.

Like Mr. Cöuard, Mr. Abdun-Nur has pointed out the disadvantages of action by committees. With much of this the writer agrees. Where facts are available, no committee is needed. However, where facts are absent, judgment and experience must be invoked, and here the col-

lective judgment and experience of a committee usually leads to a better result than if only one person were involved.

And finally, the writer agrees wholeheartedly with Mr. Abdun-Nur that codes should be so written as "to permit the engineer who has the skills, ability, judgment, and insight to make use of available knowledge with safety." The problem here is not in writing the code, this could be done today, but in determining which engineers should be placed in this category and should be given this freedom.

Mr. Brock has marshaled a powerful array of arguments against codes and it would appear from the first part of his discussion that he is in favor of abolishing them entirely. However, this misconception is dispelled in his next to last paragraph in which he advocates the preparation of codes which are compulsory and which are intended to serve primarily as a medium of postgraduate education for professional engineers. In this one respect, he would appear to agree with the writer's point that codes do serve to disseminate knowledge, but he obviously feels that they serve this function poorly and that any advantages obtained in this direction are outweighed by the disadvantages of codes as regulatory documents. This point can be clarified to some extent by distinguishing between "building codes" and "recommended practices." The ACI Code is written in such form as to be adopted as a legal document by local governments. For this reason, it is practically impossible to explain and justify its provisions. This is recognized as a serious deficiency and one which can be corrected in the present situation only by preparing explanatory remarks to be published separately from the code. The writer believes that this is a most desirable procedure and one that should have been followed long before now. Examples of recommended practices are in the reports of the various Joint Committees in this country. These reports contain much more information and justification and serve much better as an educational medium. They were not intended as codes and could not easily be used in such a manner by government agencies.

The writer must also take exception to Mr. Brock's distinction between "code writers" and "professional engineers." These are not two separate groups. In the United States, at least, the committee which prepares the ACI Code is made up exclusively of professional engineers and includes among its members some of the best engineering talent in the country. It is difficult to believe that these men, who contribute many hours of their time each year to the preparation of a code, which is subsequently voted on and accepted by the entire membership of the American Concrete Institute, do not believe that it serves some good purpose. It is possible, of course, that codes of the type we now have have become a habit with engineers in this country. If so, it is hoped that this paper and the several stimulating discussions that it has engendered will lead to a re-evaluation of the place of building

codes in engineering practice. The quotation from Mr. Freyssinet is particularly appropriate. One of the hypotheses which should be checked is whether codes in their present form are really the most desirable way to accomplish our goal.

Like the other discussers, Mr. Kalve feels that building codes are unduly restrictive. He also makes the pertinent point that without fundamental changes in our building codes, our progressive efforts in engineering education will be worthless. A more pessimistic but perhaps more realistic view is that unless our educational program is sharply upgraded, and our professional standards are raised accordingly, more liberal codes of the type proposed by Mr. Kalve could lead to chaos or even to tragedy.

Since Mr. Hoadley seems to be in agreement with the writer on at least the matters covered in his discussion, there is little comment that can be offered except to express appreciation for this agreement, without, however, implying any less appreciation for the contributions of the other discussers who have presented different points of view.

Discussion of a paper by Halvard W. Birkeland:

Differential Shrinkage in Composite Beams*

By D. E. BRANSON and A. M. OZELL, ALFRED L. MILLER, HOWARD H. NEWLON, JR., A. ZASLAVSKY, WILLIAM ZUK, and AUTHOR

By D. E. BRANSON† and A. M. OZELL‡

There is a need for a better understanding of the nature and range of effects of differential shrinkage in composite prestressed concrete beams. Mr. Birkeland, along with other researchers, justly points to more investigation and study being necessary in this field to enable engineers to fully understand the complex behavior of composite construction on which our design criteria must be based.

For the past 3 years the effects of differential shrinkage were studied at the University of Florida with the cooperation of the Florida State Road Department. The most recent tests just concluded have yielded much information which is being given here to supplement the work of Mr. Birkeland, and others that preceded his, in hope that it will result in a more comprehensive analysis of the problem.

The objectives of the latest tests were to determine the range of effects to be expected from differential shrinkage in composite prestressed concrete beams and to present a simple method for calculating stresses resulting therefrom for given design conditions. The effects of prestress levels, moisture conditions, and time periods between the casting dates for the two concretes were also considered.

DESCRIPTION OF TEST SPECIMENS AND TEST DATA

This study was conducted on five pairs of beams. Each pair consisted of one composite beam and one bare beam. Shrinkage specimens and control cylinders were used for each type of concrete. Table A summarizes information for the beams:

All ten beams were post-tensioned at 28 days with three straight unbonded ¾-in. Stressteel bars. The beams were 8 in. x 12 in. x 20 ft, and the slabs were 3 in. x 26 in. x 20 ft, as shown in Fig. A. The steel bars had a modulus of elasticity of 26.28×10^6 psi and yield strength (0.2 percent offset) of 147,000 psi. Fig. B shows the four composite beams in the laboratory with "control" noncomposite beams alongside.

*ACI JOURNAL, V. 31, No. 11, May 1960 (*Proceedings* V .56), p. 1123. Disc. 56-56 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V .32, No. 6, Part 2, Dec. 1960 (*Proceedings* V. 56).

†Member American Concrete Institute, Associate Professor of Civil Engineering, University of Alabama, Tuscaloosa, Ala. (Formerly graduate student at University of Florida).

‡Member American Concrete Institute, Professor of Civil Engineering, University of Florida, Gainesville, Fla.

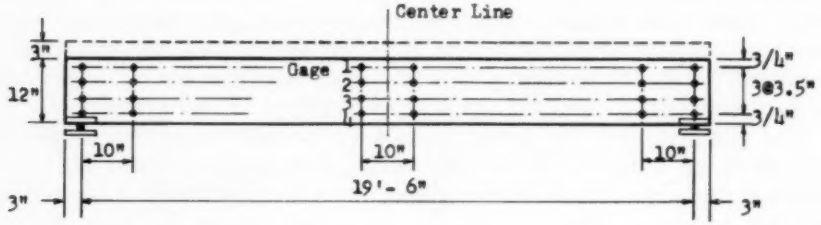
TABLE A — DATA ON TEST BEAMS

Beam No.	Concrete strength 28 days, psi	Stress level, f_{ci}'		Location	Composite beam No.	Time between prestressing and slab casting days
		percent	psi			
1,2	5030	25	1250	Laboratory	1-C	101
3,4	5030	45	2250	Laboratory	4-C	101
5,6	5030	65	3250	Laboratory	6-C	101
7,8	3760	60	2250	Laboratory	8-C	37
9,10	3760	60	2250	Field	10-C	93

A summary of the test beam data is given in Table B.

The test data for shrinkage readings for beams and slabs are presented in Fig. C and D, respectively. Fig E is a typical elastic-plus-creep strain distribution diagram (for Beam 8) and Fig. F is the total midspan camber versus time curves for all the beams.

The effect of shrinkage and creep of concrete in prestressed beams is known to be a major cause of loss of prestress, but little is known about the magnitude of the induced stresses resulting from differential shrinkage in composite beams. The term differential shrinkage is used



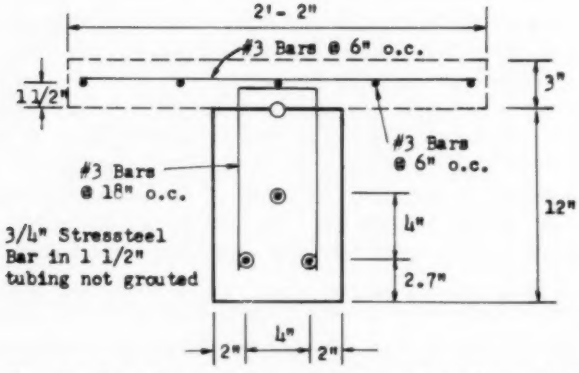


Fig. A—Bare beam with cast-in-place slab shown dotted

Fig. B—View of four composite beams in laboratory with noncomposite beams alongside

herein to refer to the differential slab shrinkage strain relative to the shrinkage-plus-creep strain of the top fiber of the precast beam.

Since the cast-in-place slab is normally placed some time after the precast beam, the unbonded shrinkage strain in the slab (shrinkage without restraint from the precast beam) usually exceeds the shrinkage-plus-creep strain at the contact surface in the prestressed beam. When the two are bonded together, an axial compressive force is induced along the top of the beam and an axial tensile force along the bottom of the slab.

In the stress analysis of composite prestressed concrete beams, the effects of shrinkage and creep can be considered in the following parts:

1. The loss of prestress resulting from the shrinkage and creep of the concrete as in ordinary prestressed concrete beams
2. The fiber stresses in both portions of the composite section resulting from differential shrinkage
3. The additional stresses in the slab, if it is reinforced, resulting from the ordinary shrinkage of a reinforced concrete element

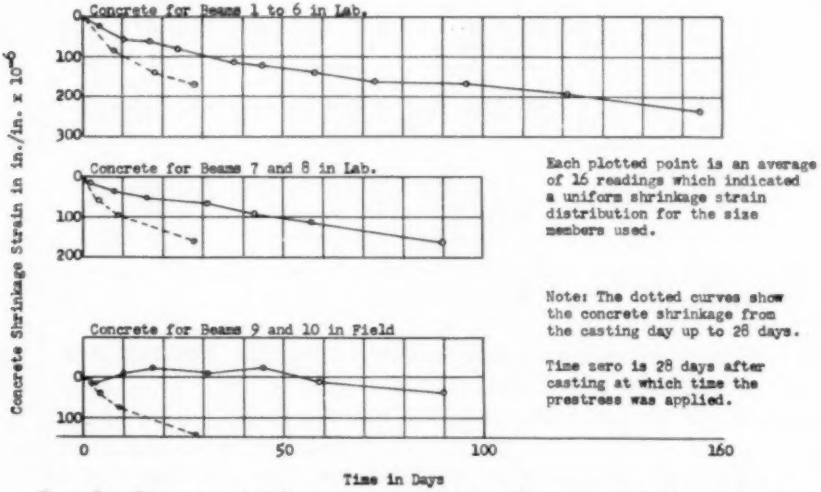

Concrete for Beams 1 to 6 in Lab.
0
100
200
300
Concrete for Beams 7 and 8 in Lab.
0
100
200
Concrete for Beams 9 and 10 in Field
0
100
Concrete Shrinkage Strain in in./in. x 10⁻⁶
0
50
100
150
Time in Days
Each plotted point is an average of 16 readings which indicated a uniform shrinkage strain distribution for the size members used.
Note: The dotted curves show the concrete shrinkage from the casting day up to 28 days.
Time zero is 28 days after casting at which time the prestress was applied.

Fig. C—Concrete shrinkage versus time for the precast beam concretes

TABLE B — SUMMARY OF DATA FOR THE TEN PRESTRESSED CONCRETE BEAMS

Beam No.	Bar No. (1 ∘ 2 ∘ 3)	Jack load	Actual load using electrical gages[a]	Actual total load, P	Measured eccentricity at beam end e_o	Actual eccentricity at midspan e_m	Total Pe_o	Total Pe_m	Actual eccentricity of c.g.s. $e = Pe/P$[b]		Initial computed stresses				σ_{max}[c]
											End		Midspan		Percent f_{ci}'
									End	Midspan	σ_T	σ_B	σ_T	σ_B	
(1)	(2)	(3) kips	(4) kips	(5) kips	(6) in.	(7) in.	(8) in.-k.	(9) in.-k.	(10) in.	(11) in.	(12) psi	(13) psi	(14) psi	(15) psi	(16)
1	1	21	18.6	57.9	-.56	-.41	125.0	133.5	2.16	2.31	+28	−1352	−250	−1059	26.9
	2	21	19.8		3.41	3.56									
	3	21	19.5		3.48	3.62									
2	1	21	22.1	65.1	-.62	-.47	131.6	140.9	2.02	2.16	−17	−1470	−291	−1182	29.2
	2	21	20.8		3.38	3.52									
	3	21	22.2		3.38	3.52									
3	1	36	33.2	101.5	-.49	-.25	223.4	247.8	2.20	2.44	+73	−2394	−119	−2190	47.5
	2	36	33.0		3.51	3.75									
	3	36	35.3		3.51	3.75									
4	1	36	37.2	99.9	-.60	-.25	190.6	225.9	1.91	2.26	−86	−2190	−219	−2048	43.6
	2	36	31.0		3.38	3.75									
	3	36	31.7		3.41	3.75									
5	1	50	49.7	142.1	-.60	-.25	283.8	334.1	2.00	2.35	−55	−3188	−106	−3130	63.5
	2	50	47.7		3.38	3.75									
	3	50	44.7		3.41	3.75									
6	1	50	46.7	139.5	-.65	-.25	283.3	336.4	2.03	2.41	−28	−3156	− 64	−3112	63.8
	2	50	46.6		3.41	3.75									
	3	50	46.2		3.35	3.75									
7	1	36	35.7	93.6	-.51	-.25	184.5	208.2	1.97	2.22	−48	−2083	−242	−1875	55.4
	2	36	29.2		3.52	3.75									
	3	36	28.7		3.48	3.75									
8	1	36	26.2	87.4	-.51	-.25	200.6	222.9	2.30	2.55	+108	−2106	− 95	−1891	56.0
	2	36	28.7		3.48	3.75									
	3	36	32.5		3.51	3.75									
9	1	36	30.1	90.0	-.34	-.25	209.8	217.5	2.33	2.41	+129	−2187	−154	−1890	58.3
	2	36	30.0		3.70	3.75									
	3	36	29.9		3.63	3.75									
10	1	36	28.5	91.6	-.28	-.25	220.6	229.6	2.41	2.51	+170	−2265	−106	−1975	60.4
	2	36	30.2		3.54	3.75									
	3	36	32.9		3.70	3.75									

[a] These values were used in the calculations.
[b] The midspan eccentricity is greater than at the beam end because of camber.
[c] The 28-day cylinder strengths were 5030 psi for Beams 1 through 6 and 3760 psi for Beams 7 through 10. The beams were prestressed at 28 days.

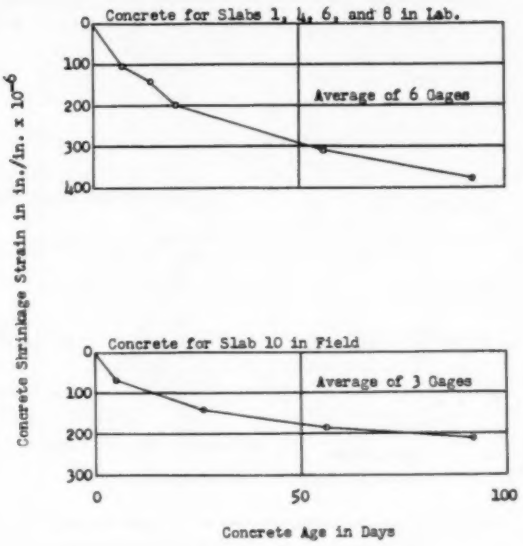


Fig. D—Concrete shrinkage versus time for the slab concretes

The ordinary shrinkage and creep of the beam and the shrinkage stresses in the slab, if reinforced, are separate effects which must be considered in addition to the differential shrinkage effect. This can be seen from the fact that if the slab were not reinforced and there were no shrinkage or creep in the precast beam, the differential effect would still occur because of the shrinkage or shortening of the slab concrete.

The most important variables influencing the effect of differential shrinkage are the following:

1. Relative casting dates
2. Surrounding moisture conditions
3. Magnitude of prestress
4. Size of member (both in span and cross section)
5. Amount of reinforcement in the slab
6. Concrete age when prestress is applied
7. Relative quality of the two concrete mixes

Since the study of the effect of different concrete mixes as a variable (No. 7) would require a relatively large number of specimens, this variable was not evaluated in the experimental work. It is clear, however, that the effect of concrete quality on differential shrinkage would be in direct proportion to its effect on plain concrete shrinkage in the two component parts of the composite beam and on concrete creep in the prestressed beam. The effect of concrete quality on shrinkage and creep has been reported in the literature.

The age when the sustained prestress force is applied (No. 6) affects differential shrinkage in accordance with its effect on the net creep

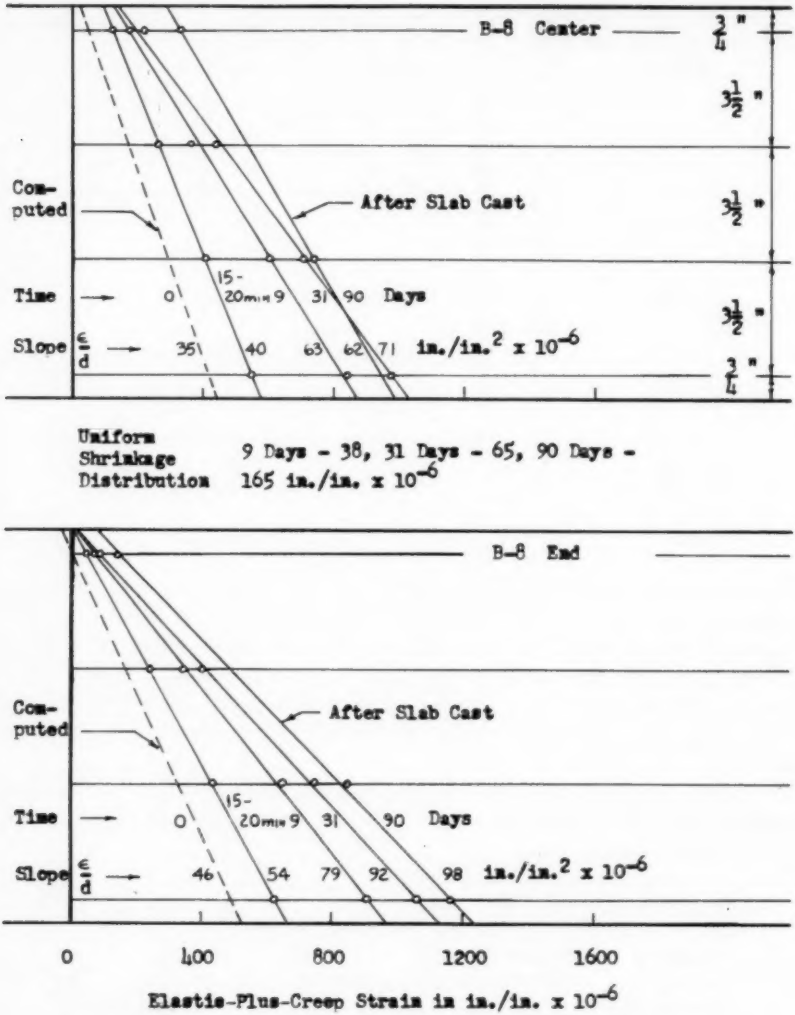


Fig. E—Elastic-plus-creep strain distribution diagrams (Beam 8)

strain of the top fiber of the precast beam following the casting of the slab.

A reinforced concrete slab will undergo smaller shrinkage strains than a plain concrete slab (No. 5) because of the restraint of the slab steel to shortening, and thus the differential shrinkage effect will be smaller when the slab is reinforced.

The differential shrinkage strains are affected by the dead-load creep strain and thus the size of member (No. 4) of the cast-in-place slab

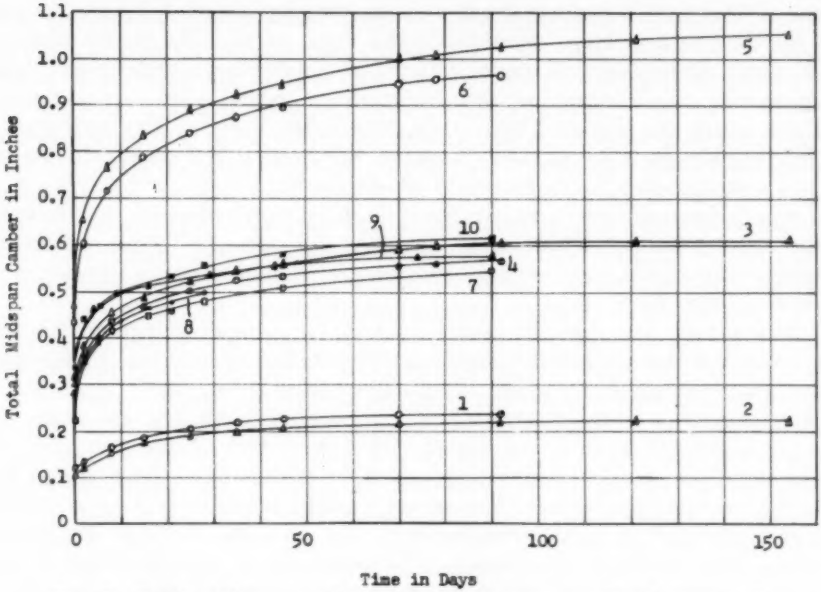


Fig. F—Total midspan camber (from dial gage readings) versus time

as well as the precast beam. The difference in the differential shrinkage effect at midspan and at the supports is reported for the test beams.

The magnitude of prestress (No. 3) is seen to affect differential shrinkage in accordance with its effect on the creep strain of the top fiber of the precast beam.

In most cases, however, the predominant variables affecting the magnitude of the differential shrinkage stresses are undoubtedly the relative casting dates (No. 1) and the surrounding moisture conditions (No. 2).

It was the purpose of this study to examine the relative effects of these variables on differential shrinkage and to present a simple method for calculating differential shrinkage stresses in both portions of the composite prestressed concrete beam. Experimental data are included showing the effect of the first three variables, and experimentally determined differential shrinkage coefficients are presented for a wide range of differences in casting schedules and relative humidities. The differential shrinkage force for each composite beam was determined experimentally by means of a beam calibration for the purpose of corroborating the analytical method set forth and the experimentally determined coefficients.

DIFFERENTIAL SHRINKAGE EFFECT

It is necessary to understand the nature of the strains occurring in both portions of the composite section as a result of concrete shrinkage and creep in order to explain the calculations needed for determining

the differential shrinkage force and the corresponding stresses. As the slab tends to shorten, relative to the stem of the composite section, an axial compressive force is induced along the top of the stem. The resistance of the stem to slab shortening produces an equal tensile force along the bottom of the slab. The total effect of the differential slab shrinkage is to cause downward bending of the composite beam and a strain distribution, such as shown in Fig. Gc.

The magnitude of these strains will vary from supports to midspan because of the dead load creep strain at the top of the precast beam. Hence the differential shrinkage stresses will be smaller at midspan than at the beam ends.

The strain distribution values in Fig. Gc, when multiplied by the modulus of elasticity of the stem concrete, lead at once to the differential shrinkage stress distribution in the stem of the composite beam since it is assumed that the stress-strain relationship for the concrete for such small stress levels is linear. However, this is not the case in the slab because the actual slab strain is a net shortening from shrinkage and the shrinkage strains themselves are not accompanied by concrete stress. Rather, it is the resistance of the stem to the slab shrinkage which produces the slab stresses. The strain corresponding to this stress, which is the same as the decrease in the unbonded shrinkage strain resulting from the stem restraint, is shown in Fig. Gd.

The reinforcing steel also provides a restraint on slab shortening and produces tensile stresses in the concrete, but this can be considered separately provided the coefficient defining the magnitude of the differential shrinkage effect has been properly determined for such reinforcement.

An important deduction must be made regarding the strain distribution in both portions of the composite section. As the differential shrinkage occurs, the entire composite beam bends downward. Assuming perfect bond between the two sections, the curvature of the composite section is given by

$$\text{Curvature} = 1/R = \varepsilon/y$$

Hence it follows that the strain distribution is linear across the composite section. This compressive strain in the slab is the actual slab strain from shrinkage, but it does not reflect the decrease in the unbonded slab strain caused by the stem restraint, which corresponds to the differential shrinkage stresses in the slab. Therefore, the stress distribution in each section and its corresponding strain distribution (Fig. Gd) do not necessarily have the same slope but depend rather on the individual properties of the respective cross sections. This is obvious from the fact that the differential shrinkage force is the same on both portions of the composite section, and hence the strain distribution diagram slope for each part, in general, will not be the same.

A question might be asked at this point about the need for a transformed composite section to be used in the determination of the differential shrinkage stresses to allow for different moduli of elasticity in the two sections. There is no external bending moment on the composite section as a result of differential shrinkage and hence no internal resisting couple. The differential shrinkage force acting in one direction on the top of the stem and in the opposite direction on the bottom of the slab exert bending moments on their respective sections. These moments are different when the two section depths are different. Each moment is resisted by its respective section and not by a combined transformed composite section, since no moment acts on the composite section itself as a result of differential shrinkage. This also suggests that the stress and strain distributions in each portion of the composite section depend on the properties of that section alone and that the stress and strain distribution diagrams do not necessarily have the same slope. This differs from the assumption made by Evans and Parker and by the author that the strain distribution diagram corresponding to the differential shrinkage stresses is linear across the composite section. The consequence is a difference in the computed top fiber stress of the slab and the bottom fiber stress of the stem.

In the presentation of a method for calculating differential shrinkage stresses it is desirable to use a variable coefficient which is independent of the two cross sections used in the composite beam. For this reason the differential shrinkage coefficient D, as defined, is used instead of a simpler value such as one of the extreme fiber strains. The differential shrinkage coefficient is defined as the difference in the shrinkage and creep strains of the two sections at the contact surface if they were not bonded. It is also the sum of the actual strain of the bottom of the slab (this is equal to the actual top fiber strain of the stem assuming perfect bond) and the tensile strain of the bottom of the slab caused by the stem restraint.

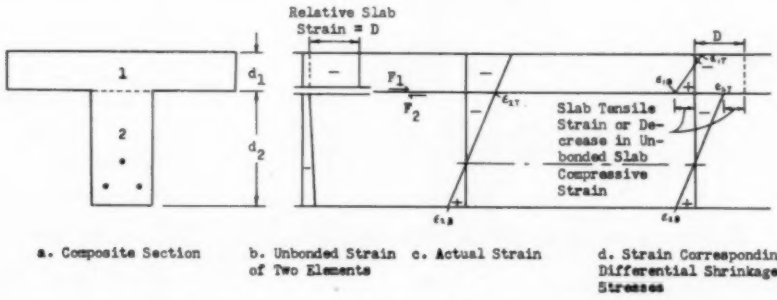


Fig. G — Composite section and strain distributions resulting from concrete shrinkage and creep (from the time of slab casting)

The differential shrinkage stresses and strains refer to those resulting from differential shrinkage alone (the differential effect of shrinkage of the cast-in-place concrete and the shrinkage-plus-creep of the top fiber of the prestressed concrete, while the two are bonded together). The assumption is made that the slab shrinkage exceeds the shrinkage-plus-creep strain of the top fiber of the stem. Otherwise the types of strains (tensile and compressive) would be reversed.

Subscripts 1 and 2 refer to the cast-in-place and precast sections respectively. Subscripts T and B refer to the top and bottom fibers of each section respectively.

After the two sections are bonded together, the differential shrinkage induces a compressive force F_2 along the top fiber of the stem. This is opposed by a tensile force F_1 along the bottom fiber of the slab.

Referring to Fig. G,

$\Sigma M = 0$, F_1 coincides with F_2

$\Sigma H = 0$, $F_1 = F_2 = F$ = Differential shrinkage force

D = differential shrinkage coefficient (experimentally determined)

= (unbonded shrinkage strain of the bottom fiber of the slab) — (unbonded shrinkage-plus-creep strain of the top fiber of the stem)

= $(\varepsilon_{1B}) + (\varepsilon_{2T})$ = (tensile strain in the bottom fiber of the slab caused by the restraining action of the stem on the relative slab shortening; that is, differential shrinkage) + (compressive strain in the top fiber of the beam caused by the same restraining action)

$$D = \varepsilon_{1B} + \varepsilon_{2T} \quad (27)$$

$$\sigma = \varepsilon E \quad (28)$$

$$\sigma = F/A + Fec/I \quad (29)$$

Hence

$$F_1 = \frac{\sigma_{1B}}{1/A_1 + e_1c_1/I_1} = \frac{\varepsilon_{1B}E_1}{1/A_1 + e_1c_1/I_1} \quad (30)$$

$$F_2 = \frac{\sigma_{2T}}{1/A_2 + e_2c_2/I_2} = \frac{\varepsilon_{2T}E_2}{1/A_2 + e_2c_2/I^2} \quad (31)$$

Since

$$F_1 = F_2 = F$$

$$\frac{\varepsilon_{1B}}{\varepsilon_{2T}} = \frac{E_2}{E_1}\,\frac{1/A_1 + e_1c_1/I_1}{1/A_2 + e_2c_2/I_2} \quad (32)$$

and

$$D = \varepsilon_{1B} + \varepsilon_{2T} = \varepsilon_{2T}\left(\frac{\varepsilon_{1B}}{\varepsilon_{2T}} + 1\right) = \varepsilon_{2T}\left(\frac{E_2}{E_1}\,\frac{1/A_1 + e_1c_1/I_1}{1/A_2 + e_2c_2/I_2} + 1\right) \quad (33)$$

or

$$D = \varepsilon_{1B} + \varepsilon_{2T} = \varepsilon_{1B}\left(\frac{\varepsilon_{2T}}{\varepsilon_{1B}} + 1\right) = \varepsilon_{1B}\left(\frac{E_1}{E_2}\,\frac{1/A_2 + e_2c_2/I_2}{1/A_1 + e_1c_1/I_1} + 1\right) \quad (34)$$

Rewriting Eq. (33) and (34),

$$D = \varepsilon_{2T}\left[\frac{(1/A_1 + e_1c_1/I_1) + (1/A_2 + e_2c_2/I_2)\,E_1/E_2}{(1/A_2 + e_2c_2/I_2)\,E_1/E_2}\right] \quad (35)$$

and

$$D = \varepsilon_{1B}\left[\frac{(1/A_1 + e_1c_1/I_1)\,E_2/E_1 + (1/A_2 + e_2c_2/I_2)}{(1/A_1 + e_1c_1/I_1)\,E_2/E_1}\right] \quad (36)$$

Substituting Eq. (35) and (36) for ε into Eq. (30) and (31), respectively,

$$F = \frac{E_2D}{(1/A_1 + e_1c_1/I_1)\,E_2/E_1 + (1/A_2 + e_2c_2/I_2)} \quad (37)$$

and

$$F = \frac{E_1D}{(1/A_2 + e_2c_2/I_2)\,E_1/E_2 + (1/A_1 + e_1c_1/I_1)} \quad (38)$$

Eq. (37) and (38) are seen to be identical.

Since the differential shrinkage coefficient D was defined as the difference in the strains of the separate or unbonded concrete elements, it is independent of the sections used. Eq. (37) or (38) shows that the differential shrinkage force F, however, depends on the coefficient, the two moduli of elasticity, and the two cross sections. The coefficient D is a dimensionless quantity but the force F has the units of force. It therefore depends on the size and elastic properties of the two sections of the composite beam. For any composite T-beam design, regardless of the type of sections, the differential shrinkage force F can be determined by Eq. (37) or (38) and the corresponding stresses by Eq. (29), provided the proper coefficient D is known.

For the special case of rectangular precast and cast-in-place sections,

$$(1/A + ec/I) = 4/A \quad (39)$$

and Eq. (35) and (36) become, respectively

$$D = \varepsilon_{2T}\left[\frac{4/A_1 + (E_1/E_2)\,(4/A_2)}{(E_1/E_2)\,(4/A_2)}\right] = \varepsilon_{2T}\left[\frac{E_2A_2}{E_1A_1} + 1\right] \quad (40)$$

and

$$D = \varepsilon_{1B}\left[\frac{E_1A_1}{E_2A_2} + 1\right] \quad (41)$$

Substituting Eq. (39) into Eq. (37),

$$F = \frac{E_2D}{(E_2/E_1)\,(4/A_1) + 4/A_2} = \frac{D}{4/E_1A_1 + 4/E_2A_2} = \frac{E_1A_1E_2A_2}{4(E_1A_1 + E_2A_2)}\,D \quad (42)$$

The extreme fiber stresses in each section are

$$\sigma = F\,[\pm 1/A \pm 3/A]$$

or

$$\left.\begin{aligned} \sigma_{1T} &= -2F/A_1, & \sigma_{2T} &= -4F/A_2 \\ \sigma_{1B} &= 4F/A_1, & \sigma_{2B} &= 2F/A_2 \end{aligned}\right\} \quad (43)$$

Hence Eq. (42) and (43) provide an easy means for calculating differential shrinkage stresses when both portions of the composite beam are rectangular in cross section and the coefficient D is prescribed.

In the calculation of differential shrinkage stresses in ungrouted post-tensioned beams, it is considered entirely satisfactory to use the gross section properties for each portion of the composite beam.

Since the differential shrinkage strains are a function of time, some average value for the two moduli of elasticity is needed in the calculations. In considering ultimate differential shrinkage stresses and strains, acceptable average values are considered to be $E_{28\text{-}days}$ for the cast-in-place concrete and the approximate modulus for the precast concrete corresponding to the same time; that is, 28 days after the casting of the slab. For design purposes, the 28-day values for both concretes would usually be acceptable.

The ordinary slab tensile stress and strain resulting from the restraint of the reinforcing steel to shrinkage deformation are given by Eq. (44) and (45).

$$\varepsilon_s + \varepsilon_c = s$$

$$f_s A_s' = f_c A_c$$

Hence the expressions for concrete stress and strain become

$$f_c = \frac{np}{1 + (n-1)p} sE_c \qquad (44)$$

$$\varepsilon_c = \frac{s}{1 + 1/np} \qquad (45)$$

In considering ultimate values from these two equations, the shrinkage strain s is a function of time, and acceptable average values for E_c are $E_{28\text{-}days}$.

TEST RESULTS AND DISCUSSION

The experimental data pertaining to the differential shrinkage investigation were obtained from mechanical gage readings on the composite beams and their companion noncomposite beams at the supports, on the bottom of the cast-in-place slabs at midspan, and on the shrinkage specimens. The differential shrinkage force was also determined experimentally using a calibration between an applied axial force along the top of the beams, just before the slabs were cast, and the gage readings on the sides of the beams. The necessary accuracy required for measuring the small strains resulting from differential shrinkage was obtained through careful measurement and the use of an average of the readings from four gages for each value reported.

Following the casting of the composite slab, the stress distribution at each beam end is not changed except as a result of differential shrinkage, since the dead load stress there is zero. Hence the difference

in the composite beam strains and the noncomposite beam strains provide a direct means of determining the differential shrinkage strain distribution at the beam ends. Average values were used from the readings on both sides and both ends of each beam. Typical values (for Beam B) are reported in Fig. H.

This same procedure could not be used at midspan. The creep strains resulting from slab dead load are of the same order of magnitude as the differential shrinkage strains, and there was no way to separate the two since the companion bare beams were not subjected to the same slab dead load.

The reduction in the strain of the bottom of the slab as compared with the free shrinkage strain of the specimens provided another independent means of determining the effect of differential shrinkage. This was possible after a relatively small value [18 in. per in. $\times$ 10^{-6} for the beams in the laboratory and 10 in. per in. $\times$ 10^{-6} for the beam in the field, determined by Eq. (45)], was subtracted from the free concrete shrinkage strain to allow for the effect of the reinforcing steel in reducing the free slab shrinkage.

The term "free shrinkage" is used to refer to the plain concrete shrinkage, and "unbonded shrinkage and creep strain" to refer to the strains of the two portions of the composite beam if they were not bonded together. The differences in the free concrete shrinkage and the bonded slab shrinkage at the contact surface are shown in Fig. I for different prestess levels, moisture conditions, and relative casting schedules. These values are equal to the slab tensile strains resulting from differential shrinkage ε_{1B} after the subtraction of that portion of the strain which is due to the restraint offered by the reinforcing steel. It was shown in Fig. G that σ_{1B} was the concrete tensile stress caused by differential shrinkage or the decrease in the unbonded shrinkage strain of the bottom fiber of the slab. The differential shrinkage coefficient D was thus found at midspan using these values for ε_{1B} and Eq. (41). With these determinations of D, the corresponding differ-

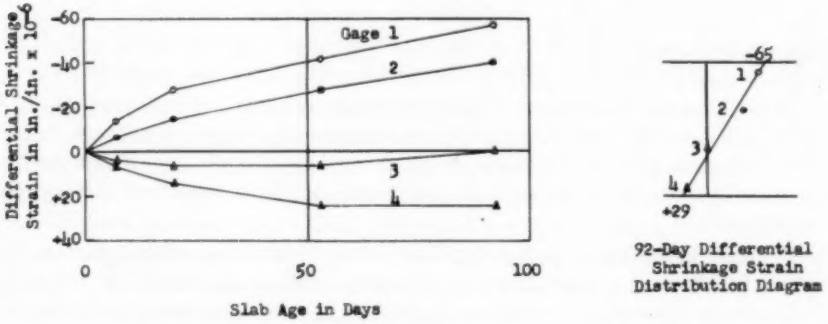


Fig. H—Differential shrinkage strains at the supports for Beam 8

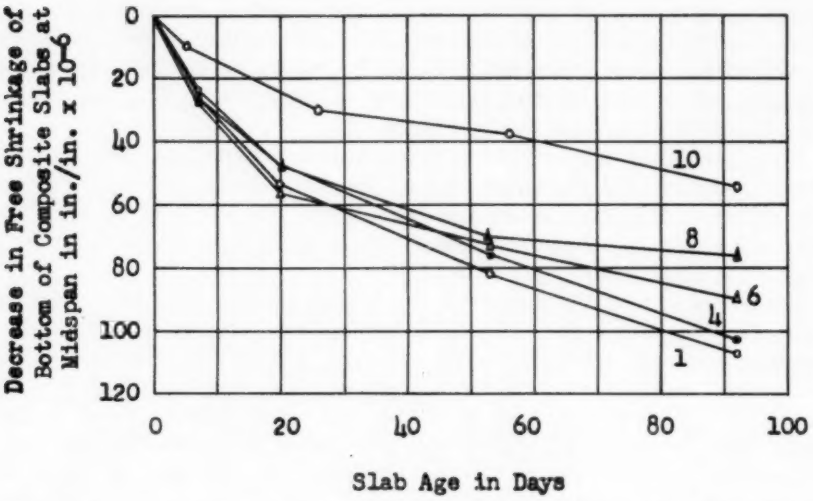


Fig. I—Decrease in free shrinkage of bottom of composite slabs at midspan versus time

ential shrinkage forces [Eq. (42)] and extreme fiber stresses [Eq. (43)] were obtained. The values for the strain at the top fiber of the precast beams, ε_{2T}, provided a means of determining the differential shrinkage coefficient D at the supports using Eq. (40). Data are in Table C.

Since the creep strain resulting from slab dead load reduces the differential strain at the contact surface, the differential shrinkage effect is markedly reduced at midspan as compared to the end values. This is shown in Fig. J for the ultimate values of D, along with the effect of different prestress levels and time periods between the casting dates for the precast beam and cast-in-place slab. The prestress level is seen to have a relatively small effect.

The effect of different relative humidities on differential shrinkage is also reported in Fig. J. With the use of the two curves in Fig. J, the proper differential shrinkage coefficient can be obtained for a wide range of relative casting schedules and relative humidities.

With regard to the use of support or midspan values for D, it is seen that the differential shrinkage stresses in the precast beam would be of interest only at midspan, but the cracking tendency of the bottom of the slab would be greater at the beam end as a result of differential shrinkage.

Several interesting points can be made with regard to the experimental values obtained and the corroboration of the data by referring to Table C. The first 15 lines of the table include pertinent data regarding the properties and characteristics of each composite beam. Line 16

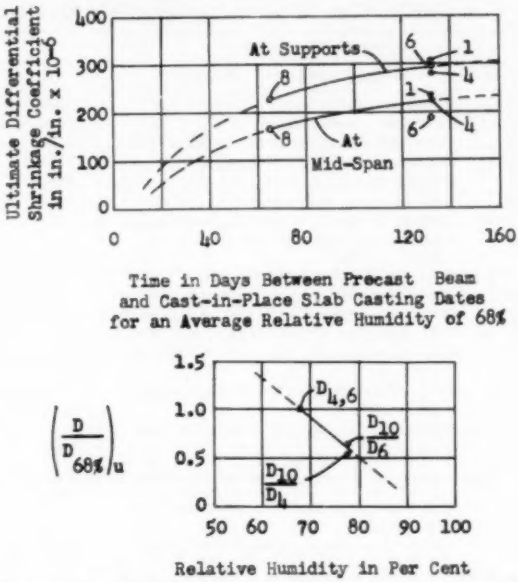


Fig. J—Curves showing the effect of different time periods between the two casting dates for composite beams, and relative humidity on the ultimate differential shrinkage coefficient, D_u

shows the differential shrinkage strain of the top fiber of the precast beams at the supports.

The differential shrinkage coefficients, forces, and stresses at the supports are shown on Lines 17 through 22 for each beam. The maximum differential shrinkage force at the end of the test period was 9650 lb in the laboratory and 5090 lb in the field. These values were obtained using the experimentally determined strains for ε_{2T} and Eq. (40), (42), and (43). The calculated bottom fiber tensile stresses in the precast beams, Line 22, are seen to compare favorably with the experimental values shown on Line 25 (maximum difference 13 percent).

Using the slopes of the strain distribution diagrams and the corresponding slopes from the gage calibration data (in the form of calibration curves for each beam—F versus slope of strain distribution diagram), the differential shrinkage force was determined for each composite beam at the end of the test period. These forces are shown on Line 27 and the calculated forces on Line 18. The reason for the experimental values being consistently lower than the calculated values (maximum 14 percent is apparently due to the fact that the concrete stiffness increased from the time of the calibration to the end of the test period, so that the force required to produce the given strains was slightly greater than that indicated by the calibration force.

The values for ε_{1B} at midspan resulting from differential shrinkage alone are given on Line 32. Using these values for ε_{1B}, and Eq. (41), (42), and (43), the differential shrinkage coefficients, forces, and stresses were found at midspan as shown on Lines 33 through 38. The maximum

TABLE C — SUMMARY OF DIFFERENTIAL SHRINKAGE DATA

Line	Explanation of values	Composite beams				
		1-C	4-C	6-C	8-C	10-C
1	Prestress level (σ_{mss}), percent f_{ci}'	25	45	65	60	60
2	Prestress level (σ_{mss}), psi	1250	2250	3250	2250	2250
3	Concrete age when prestressed, days	28	28	28	28	28
4	Concrete age when slab cast, days	129	129	129	65	121
5	Precast beam concrete, f_c', psi	5030	5030	5030	3760	3760
6	Cast-in-place slab concrete, f_c', psi	4090	4090	4090	4090	3840
7	Length of test, composite beams, days	92	92	92	92	92
8	Average temperature for period, Line 7, deg F	74	74	74	74	60
9	Average RH for period, Line 7, percent	68	68	68	68	77
10	Gross rectangular area, slab (A_1), sq in.	78	78	78	78	78
11	Gross rectangular area, stem (A_2), sq in.	96	96	96	96	96
12	Slab modulus, E_1, 28 days, psi $\times 10^6$	4.2	4.2	4.2	4.2	4.1
13	Stem modulus, E_2, 121 + 28 = 149 days, psi $\times 10^6$					4.6
14	Stem modulus, E_2, 65 + 28 = 93 days, psi $\times 10^6$				4.4	
15	Stem modulus, E_2, 129 + 28 = 157 days, psi $\times 10^6$	4.9	4.9	4.9		
	Remainder of table (except for D_s, Lines 43 and 44) refers to end of test period, Line 7.					
16	Top fiber stem strain ϵ_{2T} at supports in. per in. $\times 10^{-6}$	82	74	79	65	46
17	D S coefficient, D, at supports [Using Line 16 and Eq. (40)], in. per in. $\times 10^{-6}$	199	180	192	148	110
18	D S force, F, at supports [Using Line 17 and Eq. (42)], lb	9650	8720	9300	6860	5090
19	At supports, $\sigma_{1B} = 4F/A_1$, psi	495T	447T	477T	352T	261T
20	At supports, $\sigma_{1T} = 2F/A_1$, psi	248C	224C	239C	176C	131C
21	At supports, $\sigma_{2T} = 4F/A_2$, psi	402C	363C	387C	286C	212C
22	At supports, $\sigma_{2B} = 2F/A_2$, psi	201T	182T	194T	143T	106T
23	$\sigma_{2T} = \epsilon_{2T}E_2$, psi. Arithmetic check on Line 21.	402C	363C	387C	286C	212C
24	Bottom fiber stem strain, ϵ_{2B}, at supports, in. per in. $\times 10^{-6}$	47	42	35	29	23
25	Comparison with Line 22. Using Line 24 and $\sigma_{2B} = \epsilon_{2B}E_2$, psi	230T	206T	172T	128T	106T
26	Slope of D S strain distribution diagrams at supports, in. per in. $\times 10^{-6}$					
27	Comparison with Line 18 using gage calibration and Line 26, lb	9000	8100	8000	6600	4800

differential shrinkage force at the end of the test period was 7300 lb in the laboratory and 3610 lb in the field.

The stress magnitudes that are of interest are shown on Lines 40 through 42 for each beam at the end of the test period. The maximum values for these were as follows: at the supports, $\sigma_{1B} = 571$ psi T; at midspan, $\sigma_{2B} = 152$ psi T, and $\sigma_{2T} = 304$ psi C. The subscripts 1 and 2 refer to the slab and stem, respectively, and T and B to the top and bottom fibers, respectively. It is seen that the corresponding ultimate values for these may be important design considerations, since they could produce cracking or excessive compressive stresses when coupled with dead and live load stresses. There was no evidence of the slab cracking at the end of the test period, although it is believed that the stresses in the bottom of the slabs at the beam ends were near the tensile strength of the concrete.

TABLE C (cont.) — SUMMARY OF DIFFERENTIAL SHRINKAGE DATA

Line	Explanation of values	Composite beams				
		1-C	4-C	6-C	8-C	10-C
28	Free slab shrinkage (Avg of shrinkage specimen values), in. per in. $\times 10^{-6}$	377	377	377	377	210
29	Average strain of slab bottom at midspan, in. per in. $\times 10^{-6}$	270	274	287	299	155
30	Differential strain of slab bottom at midspan (Line 28 − Line 29), in. per in. $\times 10^{-6}$	107	103	90	78	55
31	Portion of Line 30 attributed to restraint of slab steel and not a part of D S using Eq. (45), in per in. $\times 10^{-6}$	18	18	18	18	10
32	Bottom fiber slab strain ϵ_{1B} at midspan (Line 30 − Line 31), in. per in. $\times 10^{-6}$	89	85	72	60	45
33	D S coefficient D at midspan [Using Line 32 and Eq. (41)], in. per in. $\times 10^{-6}$	152	144	122	107	78
34	D S force F at midspan [Using Line 33 and Eq. (42)], lb	7300	6970	5900	4920	3610
35	At midspan, $\sigma_{1B} = 4F/A_1$, psi	374T	357T	302T	252T	185T
36	At midspan, $\sigma_{1T} = 2F/A_1$, psi	187C	179C	151C	126C	93C
37	At midspan, $\sigma_{2T} = 4F/A_2$, psi	304C	290C	246C	205C	150C
38	At midspan, $\sigma_{2B} = 2F/A_2$, psi	152T	145T	123T	103T	75T
39	$\sigma_{1B} = \epsilon_{1B}E_1$, psi. Arithmetic check on Line 35	374T	357T	302T	252T	185T
40	Bottom fiber slab stress at beam ends (σ_{1B} = Line 19 − Line 31 × Line 12), psi	571T	523T	553T	428T	302T
41	Bottom fiber stem stress at midspan (σ_{2B} = Line 38), psi	152T	145T	123T	103T	75T
42	Top fiber stem stress at midspan (σ_{2T} = Line 37), psi	304C	290C	246C	205C	150C
43	Ultimate D S coefficient at supports* (D_u = Line 17/0.65), in. per in. $\times 10^{-6}$	306	277	294	228	169
44	Ultimate D S coefficient at midspan (D_u = Line 33/0.65), in. per in. $\times 10^{-6}$	234	222	188	165	120

*Assumes shrinkage was 65 percent complete at the end of the test period shown on Line 7.

The estimated ultimate values for D from Lines 17 and 33 are shown on Lines 43 and 44. These were also the values used in Fig. J. The ultimate differential shrinkage stresses would be increased in the same proportions.

With reference to the most important variables affecting differential shrinkage, the relative casting dates and surrounding moisture conditions were found to have the greatest effect. These are shown in Fig. J. The effect of magnitude of prestress was found to be relatively small, as can be seen also in Fig. J and in Table C.

The size of member is thought to have little effect in a general sense, since the prestress force is usually adjusted in accordance with the dead load stress. The tensile stress in the slab resulting from the restraint of the slab reinforcing to concrete shrinkage was found

to be about 13 percent of the total tensile stress in the bottom fiber of the slab. Hence, for a plain concrete slab, the differential shrinkage force would be greater (about 13 percent for the beams used). The ratio of steel area to concrete area for the slabs was 0.00705. For more heavily reinforced slabs, the differential shrinkage coefficient would be correspondingly reduced.

The effect of concrete age when the prestress force is applied and concrete quality on differential shrinkage were not included in this investigation. Their effects, which could undoubtedly be appreciable in some cases, are thought to be relatively small in most instances, at least within the limits of the accepted design practice. The main effect of time on differential shrinkage, other than total age, is the period of time between the casting of the stem and slab concretes.

The only difference in the differential shrinkage coefficient for pretensioned beams as compared to post-tensioned beams would be in accordance with the concrete age at stress release and the resulting creep history. However, this is believed to be negligible in most practical cases.

SUMMARY

The effects of differential shrinkage in composite construction were found to be significant, particularly in relatively low humidity regions. A simple analytical method was presented for the calculation of differential shrinkage stresses. Values were given for the necessary differential shrinkage coefficient covering a wide range of the principal variables involved. The effects of different prestress levels, member sizes, amount of slab reinforcement, concrete age when the prestress force is applied, and quality of the two concretes were found to be relatively small compared to the effects of different moisture conditions and time periods between the casting of the two concretes.

By ALFRED L. MILLER*

Differential shrinkage causes warping of structural members together with stresses and deflections that in the past have been ascribed to plastic flow. Current studies[3,4] demonstrate that eccentric shrinkage is largely responsible for the phenomenon and its associated effects. The paper is a significant contribution and extension of the subject and provides the means whereby warping and its effects can be anticipated.

It is noted that the analysis and theory are applicable to all composite members of which concrete is a component including concrete-steel, concrete-wood, and new concrete in conjunction with cured concrete members. The basis for accurate prediction of the deformations and stresses should preclude many of the difficulties that have arisen with

*Member American Concrete Institute, Professor of Mechanics and Structures, University of Washington, Seattle, Wash.

regard to bonding, keying, sagging, and adverse autogenous prestressing.

An alternative development of the relationships and formulations of the expressions which produces identical results and conclusions is suggested. The author's notation is followed with the addition of $\bar{y}$, the distance from the centroid of the beam to the neutral axis of autogenous flexure, and warping curvature, k.

When the concrete of the flange has become integrally keyed or bonded to the beam, it continues to shrink and thereby induces compressions, F, in the beam and tension, F, in itself. These resultant stresses are equal and opposite as shown in Fig. K (c). Considering the translational and rotational components of stresses in the flange and in the beam as shown by Fig. 3 (e) of the paper, they cause the deformation and strains as indicated by Fig. K (b). Let ε represent the free shrinkage strain after integration and k the curvature that is produced.

The curvature, k, is common to beam and flange portions of the integrated section. In accordance with the theory of flexure,

$$k = M_f/E_fI_f = M_b/E_bI_b \text{ so } Fe_f/E_fI_f = Fe_b/E_bI_b. \text{ Also } e_{bf} = e_b + e_f.$$

Therefore

$$e_b = e_{bf}E_bI_b/(E_bI_b + E_fI_f) \quad (46)$$

The intensity of normal stress at any point on the beam portion of the section is $f = F/A_b \pm Fe_by/I_b$ with respect to the beam centroid. At the neutral axis, $f = 0$ so

$$\bar{y} = I_b/A_be_b \quad (47)$$

At the centroid of the flange the deformation is $k(e_{bf} + \bar{y})$. The unstrained length of the element is $(1 - \varepsilon)$ plus the strain, $F(1 - \varepsilon)/E_fA_f$, plus the deformation equals 1. Since $F = kE_fI_f/e_f$ these relationships are combined to find the curvature in terms of the free shrinkage strain.

$$k = \frac{\varepsilon}{I_f(1 - \varepsilon)/A_fe_f + e_{bf} + \bar{y}} \quad (48)$$

Knowing k, the resultant normal stress which is also the resultant shear along the interface is calculated by

$$P = kE_bI_b/e_b = kE_fI_f/e_f \quad (49)$$

The maximum deflection of a simple span due to warping is found as in Reference 3.

$$\Delta = kL^2/8 \quad (50)$$

After the preceding items have been determined, calculation of the intensity of normal stresses is accomplished by the conventional combined stress equation applied to the beam and flange portions about the respective centroidal axes. Examples 1 and 2 of the author analyzed

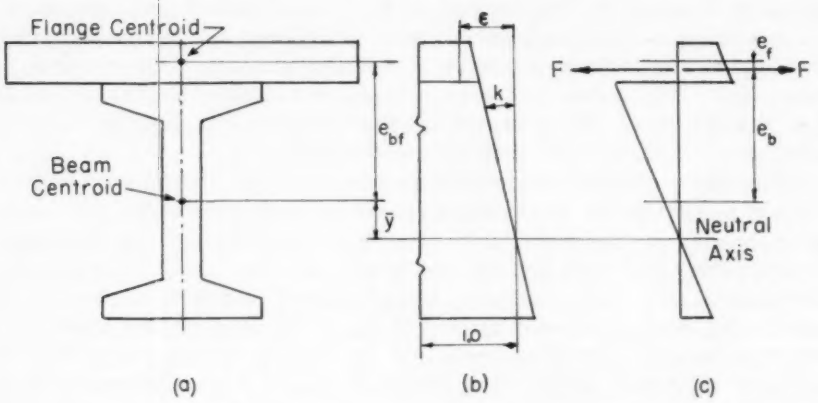


Fig. K—Section, strains, and stresses

by the above procedure yield results that are identical with those calculated in the paper.

Serious effects in multispan structures arising from warping due to concrete shrinkage have been observed and analyzed by Wilburn.[4] It is suggested that similar conditions exist in composite members that deserve further study in accordance with the findings submitted in the paper and this discussion.

It is re-emphasized that transmission of the resultant force *F*, on the interface may be provided for near the ends of each span in which case the intensity of longitudinal shear throughout the remainder of the span length is negligible.

In addition to warping, the induced prestress can have adverse effects and should be anticipated by the designer especially where continuity is involved.

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By HOWARD H. NEWLON, JR.*

Mr. Birkeland is to be congratulated for focusing attention on an area where much speculation has prevailed but in which published information is lacking. While his paper deals primarily with a prestressed beam carrying a concrete slab, the situation is analogous to that of a steel beam carrying a concrete slab. This latter type of

*Member American Concrete Institute, Highway Research Engineer, Virginia Council of Highway Investigation and Research, Charlottesville, Va.

structure of course is quite common and, as the writer became aware while conducting thesis research,[5] the effect of differential shrinkage in this case has been discussed at length in the literature. Because the majority of this discussion has been in the German literature, American engineers by and large have not had the full benefit of it. Thus a brief summary of previous work to show the value of Mr. Birkeland's work appears justified.

Apparently Morsch in 1943[6] was the first to study the problem. Utilizing the assumption of an eccentrically loaded column he developed the shrinkage force exactly as given in the author's Eq. (11) and stresses as given in the author's Eq. (15) and (16). It should be noted that this procedure was also used by Viest, Fountain, and Singleton[7] in their book published in 1958.

In 1948 Haulena[8] proposed another method for accomplishing the calculation. In the notation of the author's Fig. 3(b), using a unit length, Haulena reasoned that the active or shrinkage force (P, acting to the right) occurred only in the concrete, whereas the related reaction force (P', acting to the left) acted both as a normal force and a moment on the composite section. Using ε to signify the displacement due to unrestrained shrinkage, he reasoned that a force P was required to restrain this movement. By letting ε_1 signify the displacement due to a force $P' = 1$ acting on the composite cross section, he stated that

$$\varepsilon_1 P' + \varepsilon = 0 \qquad (51)$$

Applying a unit load $P' = 1$, he equated the work done by the unit load in straining the concrete by an amount ε_1 to the internal strain energy, recognizing three strains; viz., a strain at the centroid of the concrete, a strain at the centroid of the steel, which was also common to the composite section, and a rotational strain. From this he found:

$$P' = \frac{E_t \varepsilon}{\frac{1}{A_t} + \frac{e_{ct}^2}{I_c} + \frac{1}{A}} \quad \text{and} \quad M = P e_{ct} \qquad (52)$$

Comparison of this equation with Eq. (11) of the author will show that a much smaller force results from Eq. (52) than from Eq. (11). The corresponding steel stress at the interface would be

$$f = + \frac{P}{A_t} - \frac{P}{A_t + nA_b} - \frac{M y_{ct}}{I_c} \qquad (53)$$

The stresses resulting from this and corresponding equations are considerably lower than would result from application of the method of either Morsch or the author. Haulena's approach is somewhat intuitive and some question might be raised concerning the consideration of the direct strain based upon the concrete section in addition to the rotational and direct strains based upon the composite section.

In 1950, Frohlich[9] approached the problem from the standpoint of the individual elements of the section, with the restriction that the

two were forced to act together. Using the author's Fig. 3(b) with the exception that the shear forces P are placed at the interface, Frohlich considered four strains: ε_f, the direct strain in the concrete; ε_f', the rotational strain in the concrete; ε_b the direct strain in the steel; and ε_b', the rotational strain in the steel. Evaluating the strains and moment.

$$\varepsilon_f = \frac{P}{A_f E_f}, \quad \varepsilon'_f = \frac{M_f y_{f1}}{E_f I_f}, \quad \varepsilon_b = \frac{P}{A_b E_b} \qquad (54)$$

$$\varepsilon'_b = \frac{M_b y_{b1}}{E_b I_b}, \quad M = P e_{bf} = M_f + M_b \qquad (55)$$

$$M_b = M \frac{E_b I_b}{E_f I_f + E_b I_b} = M \frac{n I_b}{I_f + n I_b} \qquad (56)$$

Likewise

$$M_f = M \frac{I_f}{I_f + n I_b}$$

For no slip at the interface,

$$\varepsilon = \varepsilon_f + \varepsilon'_f + \varepsilon_b + \varepsilon'_b \qquad (57)$$

Substituting and solving

$$P = \frac{\varepsilon E_f}{\frac{1}{A_f} + \frac{1}{A} + \frac{e_{bf}^2}{I_f + n I_b}} \qquad (58)$$

from which the stress at the interface is

$$f_b = -\frac{P}{A_b} - \frac{M_b y_{b1}}{I_b} \qquad (59)$$

Stresses at other locations could be likewise derived.

Of interest in considering this method is the similarity of Eq. (58) for determining the force and Eq. (52) from Haulena. It is apparent that the shear force computed from Eq. (52) will be considerably larger than that from the individual elements. Even though the force as determined from Eq. (58) is smaller than that obtained from Eq. (52), the stress obtained from Eq. (59) is higher than that from Eq. (53). Although it is not apparent by inspection, application of the method of Morsch and the author's Eq. (15) and (16) will result in the same stress values. It should be noted here that a paper by David and Meyerhof[10] published in 1958 contains this same approach.

In 1952, Walter [11] approached the problem in a slightly different manner in that he neglected the stiffness of the concrete slab, a procedure which affects the results by 4-5 percent for normal dimensions. The force resulting from this procedure is

$$P = \frac{E_f \varepsilon}{\frac{1}{A_f} + \frac{1}{n A_b} + \frac{e_{bf}^2}{I_f}} \qquad (60)$$

It is interesting to note here from Walter's paper that he approached the problem by expressing the stresses in terms of the centroid of the concrete since he advocated prestressing as a means of overcoming the tensile shrinkage stresses in the slab. It appears from the German literature that this was attempted but apparently the practice was not extensive.

A comparison of the various methods will show that each is based upon analysis of the beam as a column in which the entire shrinkage force is concentrated at the ends, resulting in a constant stress throughout the length of the beam. This assumption would not be valid if the force were distributed in some manner along the beam. The author states that "with ample reinforcement across the interface, this distance (that over which the force is distributed) is in the order of half to full beam depth." The writer would like to know if any measurements have been made to confirm this force distribution.

The writer[5] analyzed the situation by considering the extreme case of a uniform shrinkage force distribution rather than a force concentrated at the ends (see Fig. L).

For this case it was necessary to assume a distribution for the vertical force which would tend to separate the two elements. In the absence

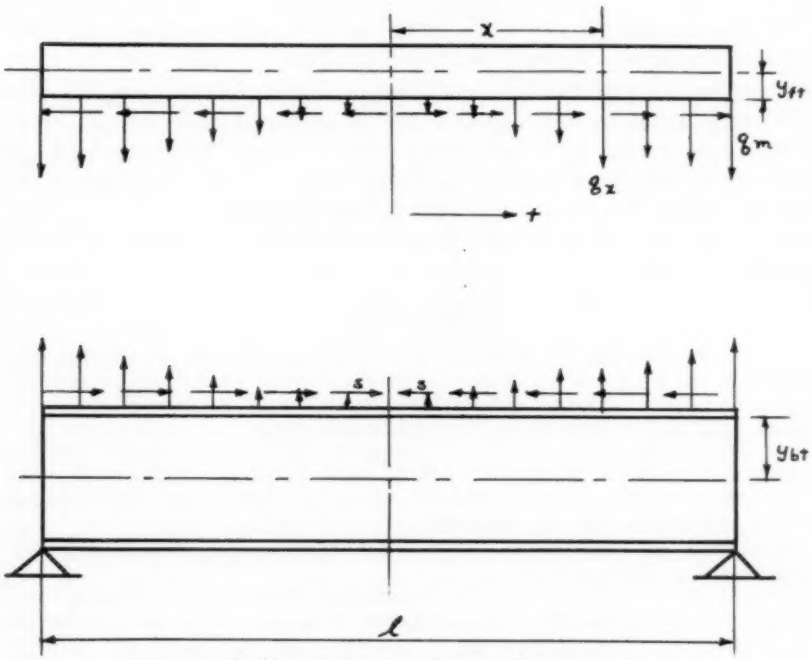

Fig. L—Shrinkage forces acting on beam and slab

of data, a linear distribution of this unit force q_m was assumed. A solution for this force in terms of p showed that for beams of normal lengths it could be neglected.

Using the four strains in a summation similar to Eq. (57), the total shear force P is given by Eq. (61)

$$P = \frac{E_{bt}}{\left[\frac{1}{A_f} + \frac{1}{nA_b} + \frac{y_{ft}^2}{I_f} + \frac{y_{bt}^2}{I_b}\right]} \quad (61)$$

With the assumption of a uniform distribution the unit force

$$P = \frac{P}{l/2} = \frac{2P}{l} \quad (62)$$

and the moment at any section

$$M_x = Py_{bt}\,(x - l/2) \quad (63)$$

From this the steel stresses at the interface would be

$$f_{bt} = \frac{-px}{A_b} - \frac{M_x Y_{bt}}{I_b} \quad (64)$$

This equation indicates a variation of stress along the length of the beam. It is recognized that this is an extreme case but it is likewise true that an analysis using a concentrated end force is also an assumption. The value of Mr. Birkeland's analysis which should be emphasized is that he did not stop with Eq. (15) and (16) but included the effect of the end couple.

The agreement of field observations with predictions is quite good and it would be interesting to learn if any measurements of strain or deflection were made at various points along the length of the beam. Observations made by the writer on a small laboratory beam indicated some distribution of the shrinkage force.

Again the author is to be congratulated for his work with this elusive problem.

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By A. ZASLAVSKY*

This interesting paper deals with a composite reinforced concrete section consisting of a precast prestressed beam and an in-situ slab.

MAGNITUDE OF DEFLECTIONS AND STRESSES

It may be noted that the calculated normal stresses produced by the differential shrinkage are not large, but should be considered at the top fiber of the beam where they are additive to the stresses produced by live load (resisted by the composite section).

The deflection is also not very great (in author's example 1: $\Delta/L = 1/1750$) but could, of course, be counteracted by initial camber.

CHECK ON STRESSES

Referring to author's Fig. 3(d) and 3(e) showing the final stress distribution due to differential shrinkage, it may be pointed out that these stresses form a self-balanced system in equilibrium (like "residual stresses") that is, they should reduce to zero, or ($F_f = F_b$):

$$F_f e_{bf} = M_f + M_b$$

Referring to author's example 1 this is readily checked:

$$108.8 \times 21.9 = 5.38 + 2377.0$$

$$2382.72 \cong 2382.38$$

EFFECTIVE SLAB WIDTH

There arises the question, what is the ratio between slab width, slab thickness, and beam width ensuring a reasonably uniform behavior of the slab across its width. In Example 1 these values are 60 in., 6 in., and 20 in., respectively. But assuming a slab width of say 100 in., it does not seem reasonable that the force $P = \varepsilon A_f E_f$ could be computed on the basis of $A_f = 6$ in. $\times$ 100 in. This question is, of course, analogous to the one encountered in the design of ordinary T sections, and excessive slab width would result in plane slab sections curved in the plan view, since the extreme longitudinal strips of the slab would be practically unrestrained and able to shrink freely.

THE COMPUTATION METHOD

The theoretical treatment by superposition is elegant and simple. This method is more generally applicable — *interalia*, also in the problem of bimetallic elements.[12] The method implies that stress transfer between slab and beam takes place at the ends only. Normal stress distribution in the sections being constant over the entire length, no shear stresses can exist along the interface (or any other parallel plane).

*Member American Concrete Institute, Associate Professor of Civil Engineering, Technion-Israel Institute of Technology, Haifa, Israel.

Attacking the problem by assuming stress transfer over the entire length of the interface (which may be the first instinctive assumption) would lead to variable normal stress distribution and it might be worth while to enlarge on the explanation why this would be incompatible with strains and curvature, or in contradiction with the plane section hypothesis.

NONPROPORTIONALITY IN COMPOSITE SECTIONS

Since the methods outlined in the paper apply to composite sections in general (including also steel beams with concrete slabs) the writer would like to emphasize the nonproportional relationship (in the elastic range) between load and stresses (and deflections) in these sections.[13,14] This is due to the fact that the first part of the load is resisted by the beam only, while the second part (live load, etc.) is resisted by the whole composite section. A logical *elastic* design should therefore be based, not upon loads producing admissible working stresses but rather upon loads producing critical stresses (at the most stressed fibers of the slab and beam, respectively). Only the second method would ensure the required safety factor against appearance of the critical stress by overloading.

In ultimate strength design, the carrying capacity of the composite section would be practically unaffected by the nonproportional relationship.

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By WILLIAM ZUK*

Over the last several years, the Virginia Council of Highway Investigation and Research has been independently conducting research on the problems of shrinkage and thermally induced stresses in composite beams. Although the writer has been particularly interested in the more variable thermal stress problem, many of these findings are valid in the domain of shrinkage also; as the case of a slab and beam each at a uniform but different temperature would be identical in nature to the shrinkage problem.

This discusser's approach differs from the author's in that the writer considered the effects of transverse slab restraint caused by adjacent beams, matching of interface curvatures, independent shrinkage of

*Professor of Civil Engineering, University of Virginia, and Highway Research Engineer, Virginia Council of Highway Investigation and Research, Charlottesville, Va.

beam and slab, as well as matching of interface strains and linearity of plane sections.

It is of particular interest to note that by means of this more general independent method, the writer confirms the author's conclusions that the entire interface forces are concentrated at the beam ends, and consist of a shear and a couple. In all prior analyses of this problem by both German and American investigators, this end couple has been overlooked. It is of importance to designers to be aware of this, as adequate coupling of slab and beam for composite behavior must provide for separating tensile forces at the ends in the form of a mechanical anchor. Thus shear connectors as deformed bars, studs, coils, and channels would provide the needed tensile anchorage; whereas keys, angles, weld buttons, and roughened surfaces would not.

Using the author's notations, the writer has evolved the following equations for the determination of the shear and moment at the interface. (m is Poisson's ratio of the slab material.)

$$\frac{(1-m^2)}{E_f A_f}\left(\frac{3}{2}\frac{M_i}{y_{fb}} - 2V_i\right) + e_f(1+m) = \frac{M_i y_{bt}}{E_b I_b} + \frac{V_i}{E_b}\left(\frac{y_{bt}^2}{I_b} + \frac{1}{A_b}\right) + e_b \quad (65)$$

$$\frac{2E_f A_f y_{fb}^2}{3(1-m^2)(V_i y_{fb} - M_i)} + y_{fb} = \frac{E_b I_b}{V_i y_{bt} + M_i} - y_{bt} \quad (66)$$

These two equations may be solved numerically for V_i and M_i. The results of these equations differ somewhat from the author's as the above equations are believed to be a bit more complete.

It may also be noted that similar interface shears and moments at the beam ends are induced in regard to thermally induced stresses if the temperature is constant along the beam length and variable through the depth of the slab and beam.

A more complete discussion of both shrinkage and thermal stresses will be presented by the writer in a forthcoming issue of the JOURNAL.

AUTHOR'S CLOSURE

The tests and studies reported by Professors Branson and Ozell were of considerable scope. It appears to the author that Branson and Ozell have assumed that the interface shear is not accompanied by any force couple, and that a vertical section that was plane before shrinkage is not necessarily plane after shrinkage. The second part is a necessary companion of the first part. The above assumption is in opposition to the author's, which was that a plane section would remain plane. The author's force couple resulted therefrom. The author stated (p. 1136) that "... There will undoubtedly be some adjustment in stresses, and they may not be linear ...", but without evidence of a section becoming non-plane, he is not ready to accept the Branson-Ozell assumption.

Another substantial difference in philosophy relates to the effective Young's modulus for the slab and the differential shrinkage. Branson

and Ozell favor using the 28-day modulus for the slab and shrinkage that is a fraction of the free shrinkage. If modulus and shrinkage are varied suitably, increasing one while reducing the other, or vice versa, it is possible to arrive at the same deflection of the composite beam from any set of values. For example, if the Norton Building test beam is analyzed with $E_f = 4000$ ksi and $E_f = 0.000174$, we obtain the same deflection as previously, $\Delta_{max} = 0.440$ in. As should be expected, the stresses in the beam do not change. Stresses in the slab do change, decreasing 14.7 percent in the top fiber and increasing 11.5 percent in the bottom fiber.

The difference in results can not be said to be great. However, considering that the slab concrete will grip the beam at the same time that it passes from the plastic to the solid state, the author favors using a low effective Young's modulus for the slab concrete, such as the 1000 ksi of the examples. Also, with beams that are substantially aged at the time the slab is cast, he favors using a differential shrinkage that approaches the free shrinkage of the slab concrete, such as the 0.00045 of Example 1. The effect of slab reinforcement, discussed by Branson and Ozell, can be included when establishing the differential shrinkage. If the longitudinal reinforcement is eccentric in the slab, the slab will have a tendency to curl when it shrinks, and this curling will also be resisted by the beam.

It should be stressed that the author's examples were related to concretes of mixes that were relatively harsh, lean, and dry, and a high density was obtained by heavy vibration. These concretes have a rather low shrinkage, and it occurs at an early age. The strength development shown in Fig. 6 is typical.

Professor Miller has contributed an elegant solution which is obtained by way of deformations. It should be emphasized that he obtains the internal force and the curvature without resorting to the transformed section and internal stresses. The simplicity appears to be due to his introduction of the neutral axis.

With the lead given by Miller, and introducing the neutral axis of the slab, the new primary equations may be summarized as follows.

Additional notation

e_b = distance from centroid of beam to resultant force on beam (i.e. eccentricity of force)
e_f = distance from centroid of slab to resultant force on slab (i.e. eccentricity of force)
$\tilde{y}_b$ = distance from centroid of beam to neutral axis of beam
$\tilde{y}_f$ = distance from centroid of slab to neutral axis of slab
R = radius of curvature (measured to neutral axis of beam)
k = curvature

Primary equations:

$$e_b = e_{bf} \frac{E_b I_b}{E_b I_b + E_f I_f} \qquad \text{(a)}$$

$$e_f = e_{bf}\,\frac{E_f I_f}{E_b I_b + E_f I_f} = e_{bf} - e_b \qquad \text{(b)}$$

$$\bar{y}_b = \frac{I_b}{A_b e_b} \qquad \text{(c)}$$

$$\bar{y}_f = \frac{I_f}{A_f e_f} \qquad \text{(d)}$$

and by similar triangles of a deformed element,

$$\frac{1}{R} = k = \frac{\varepsilon}{\bar{y}_b + e_{bf} + \bar{y}_f} \qquad \text{(e)}$$

$$M_b = k\,E_b I_b \qquad \text{(f)}$$

$$M_f = k\,E_f I_f \qquad \text{(g)}$$

$$F = V_i = \frac{M_b}{e_b} = \frac{M_f}{e_f} \qquad \text{(h)}$$

Employment of the above Eq. (a) to (h) will give the basic solution to the problem. The author prefers these equations for their simplicity in application, and appreciates very much Miller's contribution. These are transformable into those of the paper, and yield identical results. E_f is still the effective Young's modulus for the slab concrete, with a value in the general magnitude of 1000 ksi; and ε still represents the differential shrinkage, which approaches the free shrinkage of the slab concrete, and is in the order of 0.00045.

Mr. Newlon mentions that some other investigators have arrived at forces that are substantially less than those of the author. The existence of internal moments as given in the paper are substantiated by the measured deflections of the test beams. The internal forces and interface shear and moment are directly related to the moments by consideration of static equilibrium, and are thereby also substantiated.

Mr. Zaslavsky points to the question of the width of slab to include in an analysis of the shrinkage problem. The author has not had occasion or opportunity to determine the effective width of a flange, but would be inclined to use a width considerably greater than that allowed by design codes concerned with the strength of section.

Messrs. Newlon and Zaslavsky are both interested in the author's statement that stresses in the interface, due to shrinkage, exist only near the ends of the span. This can be enlarged upon.

In the test of the CTC beam (Fig. 8), deflections were measured at short time intervals with dial gages at several points along the beam. Representative readings are recorded in Table D on lines 1, 2, and 3. Changes in deflections are entered on lines 4 and 6; and relative changes, with midspan deflection reduced to unity, are entered on lines 5 and 7. Line 8 shows relative deflections computed on the basis of constant moment along the span. The close agreement between measured and computed relative deflections, lines 5 and 7 versus line 8, demonstrates that the beam was subjected to nearly constant moment

TABLE D — RELATIVE DEFLECTIONS OF CTC TEST BEAM

Type of deflection	Position of dial gages, distance from end of beam, ft			
	5	15	25	35 (midspan)
1. Age 6 hr Δ_1 in.	0.098	0.277	0.396	0.438
2. Age 2.14 days Δ_2 in.	0.212	0.575	0.799	0.875
3. Age 6.21 days Δ_3 in.	0.238	0.643	0.891	0.975
4. $\Delta_2 - \Delta_1$ in.	0.114	0.298	0.403	0.437
5. Relative $\Delta_2 - \Delta_1$	0.261	0.682	0.922	1.000
6. $\Delta_3 - \Delta_1$ in.	0.140	0.366	0.495	0.537
7. Relative $\Delta_3 - \Delta_1$	0.261	0.681	0.922	1.000
8. Computed relative deflection due to constant moment, Δ_4	0.265	0.673	0.918	1.000

through its length. Zero shear in the interface is consistent with constant moment. The data in Table D have been extracted from Arthur R. Anderson's notes on the test.

Constant moment is also a direct result of the basic approach of the paper, i.e., the application of selfcancelling external forces as shown in Fig. 1. If rigid mechanical devices had been used, capable of transmitting tensile as well as compressive normal stresses to the ends of the beam, then the entire length would be free of shear. The ends of the slab and beam may be said to take the place of such mechanical devices, and since the source of deformation is in the slab, there must be stresses in the interface near the ends to effect interaction.

The case for no shear in the interface may be viewed in still another way. The slab and beam have each been subjected only to eccentric forces, and these are evaluated in magnitude and location so that they balance each other, and so that they produce compatibility between slab and beam. Then these forces represent one solution that satisfies all conditions of the problem. If superposition and proportionality are valid assumptions, then the solution is unique.

Professor Zuk has included the effect of lateral restraint (Poisson's ratio effect) in the slab. With this it should be anticipated that the stresses in the direction of the span will be smaller. Deflections should also be smaller. Such lateral restraint will not materialize in a narrow slab with one beam, such as the test beams of the paper, but will be significant in cases of wide slabs supported by several beams.

The author appreciates the contributions by the several discussers, and feels that their material has added substantially to an understanding of the problem and to its practical solution.

Discussion of a paper by Emilio Rosenblueth and Ignacio Holtz:

Elastic Analysis of Shear Walls in Tall Buildings*

By HANNSKARL BANDEL, KIYOSHI MUTO and YUTAKA OSAWA, and AUTHORS

By **HANNSKARL BANDEL**†

The authors are to be complimented on their excellent publication. Their method of solving this rather complicated problem fascinates by its elegance. However, the method neglects the elastic shortening of the columns. The distribution of the load acting on the shear walls or on the connected frames will be increasingly influenced by this, namely with increasing height of the building and increasing stiffness of column-connecting girders, and decreasing distance of the shear wall neighboring columns and decreasing cross-area of the columns.

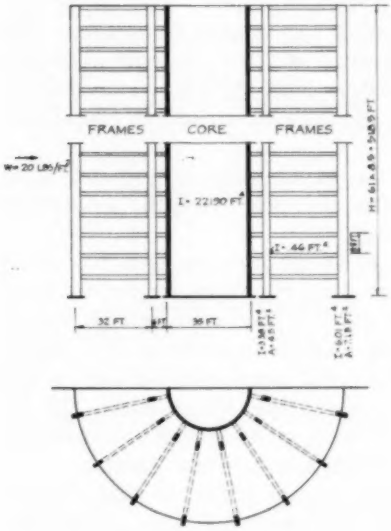

Fig. A — Data on 60-story concrete tower

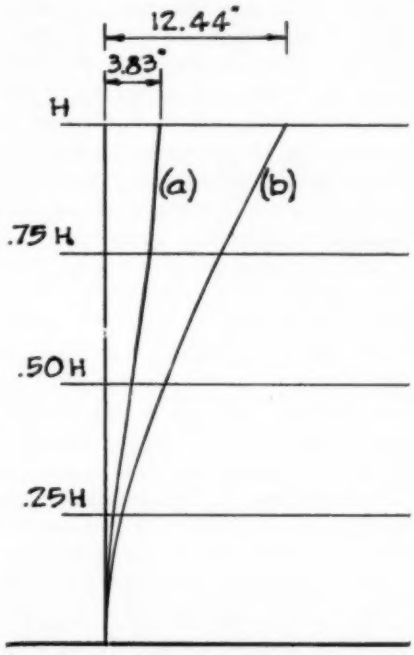

Fig. B — Horizontal deformations in building in Fig. A

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†Associate, Severund-Elstad-Krueger-Associates, Consulting Engineers, New York, N.Y.

The following example of a 60-story concrete tower building (Fig. A) is an extreme case, but it may instigate more thorough investigation in special cases. The deflection curves shown in Fig. B are obtained by a modified slope deflection method which has been developed by the writer and which will be published shortly. Rotation and vertical and horizontal deformation of the frame knot points are developed in series as a function of the height of the building. After determining the total external and internal work by integration, the constants of the series result from a partial differentiation of the work equation.

In Fig. B the horizontal deformations of the total building are compared. Line (a) represents the deflection without column deformation in the longitudinal direction and line (b) the deflection including the deformation of the columns. The deflections and the proportional load distribution resulting in moments in core and frames differ radically in the cases considered.

By KIYOSHI MUTO* and YUTAKA OSAWA†

We read with interest the paper by Messrs. Rosenblueth and Holtz since we have made some studies on a shear wall.

The paper deals with the problem of interaction between shear wall and moment resistant frames in tall buildings. The numerical method employed in the paper is useful for practical purposes. We appreciate their work because the shear wall is the most effective element in earthquake resistant structures, and therefore, analysis of a shear wall is always an important problem in structural design in a seismically active zone.

Our purpose here is not to discuss the paper itself but to introduce the outline of our study on shear walls and point out the important subjects related to their practical design.

In 1950 we studied the seismic shear distribution in uniform shear walls and open frames by using a differential equation. It is clear from this study that: (1) a tall, slender shear wall has considerable resisting capacity in the lower stories but has small capacity in the upper stories as the effective rigidity of the wall decreases owing to the accumulation of the bending deformation of the lower wall; (2) insufficient end fixity at the base of the wall gives a reduced resisting capacity because of the deformation due to foundation rotation.

In connection with the above findings, it seemed to us that the adjacent open frames both in the same and perpendicular plane can restrain the bending deformation of the wall as well as foundation rotation deformation, and this makes it possible to prevent the decrease of resisting capacity of the slender walls.

*Member American Concrete Institute, Dean and Professor of Faculty of Engineering, University of Tokyo, Tokyo, Japan.
†Member American Concrete Institute, Assistant of Earthquake Research Institute, University of Tokyo, Tokyo, Japan.

From this point of view we studied the boundary effect between shear walls and adjacent open frames from 1950 to 1955. A numerical method on this subject was first introduced in the book, *Lateral Force Distribution Coefficients and Stress Analysis for Walled Frames,*[7] in 1951, and later it was included in the paper, "Seismic Analysis of Reinforced Concrete Buildings,"[8] which was presented at the First World Conference on Earthquake Engineering in 1956. This method is to be used for relatively low, rigid walls. Another numerical method for a tall, slender wall was shortly introduced in the appendix of the above paper.[9]

In 1953 we consulted on a seismic design of the Tokyo City Hall Building. Most of the shear walls in the building were extremely slender, and it was intended to restrain the bending deformation by the boundary effect of the adjacent open frames. The model test of these frames was carried out in our laboratory and it was ascertained that the adjacent frames worked well enough to prevent the decrease of resisting capacity of the shear wall in the upper stories.

In the course of the above study we noticed that the stresses in the adjacent frames are generally larger than those in ordinary open frames by around 50 percent owing to the stress concentration. And we pointed out that in the practical design these overstresses can be decreased due to localized plastic deformation because the concrete will crack where they are overstressed.

Finally, we would like to point out two important subjects to be considered in the design of shear walls. These are: (1) the effect of foundation rotation on the resisting capacity of the wall, (2) the plastic deformation of both shear wall and adjacent open frames. Further investigation on these subjects is necessary to get more reasonable design methods.

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9. Appendix to the above paper (Separate pamphlet), pp. 105-109.

AUTHORS' CLOSURE

Dr. Bandel points out the need to consider axial deformations of the columns in some cases. He gives an example in which this factor has a decisive influence on the deflections and stresses in the frame. It is gratifying to learn that Dr. Bandel's procedure, which takes this factor into account, will be published shortly.

The writers agree with Dr. Bandel in that the effects of axial deformation merit consideration in many cases and that they simplified their

method by neglecting the effect. There are other simplifications in the paper that, under certain conditions, also deserve refinement. These are well brought up by Professor Muto and Mr. Osawa in their discussion, where they make special reference to inelastic action of the structure and the effects of foundation rotation.

The matter of inelastic action deserves considerable attention from a practical standpoint. Even in conventional design against lateral forces one often finds it uneconomical to take care of the moments induced in beams that rest on the edge of shear walls. It is advantageous then to allow the formation of plastic hinges at least in beams that rest on the shear walls. By so doing analysis is simplified, since no more than a few cycles are required to arrive at the final moments.

In a private communication Professor Kuang-Hau Chu of Illinois Institute of Technology has pointed out a numerical error in the example given in the paper. Indeed, $4p^2/\pi^2 = 1.24$ in lieu of 3.78.

Discussion of a paper by J. H. Clarke:

Method of Assessing Probable Fire Endurance of Load-Bearing Columns*

By DAVID V. ISAACS and AUTHOR

By DAVID V. ISAACS†

The paper is basically theoretical, as was Mr. Clarke's work while he was employed in the Fire Research Section of the Commonwealth Experimental Building Station. This Fire Section, in keeping with the charter of the CEBS as a whole, has always been concerned with both practice and theory, and in fact has done much of late in advising building regulatory bodies in Australia on the fire problem as a whole in buildings. Fire resistance, of course, is only one of the components of this problem, but it is of such importance, nevertheless, that we can hardly learn too much concerning it.

Research workers have established the relationship between the potential fire load of a building and the necessary minimum fire resistance of its structural elements, if the building is to resist collapse when its contents are consumed by fire. It is the function of regulatory bodies to nominate the level of fire protection they require, by specifying the levels of fire resistance, i.e., the fire-resistance ratings, which they require for the various elements of structure in the various types of buildings within their jurisdiction. A fire-resistance rating for a structural element can be firmly established only by determining its fire endurance under standard test conditions. However, where insistence upon individual test results would prove unduly onerous, it is not uncommon for regulatory authorities to nominate broad specifications for those elements of construction which they will accept as having sufficient fire resistance to qualify for various ratings. Reinforced concrete columns are often specified in this manner.

The wording of the introduction and conclusion of Mr. Clarke's paper could lead the casual reader to believe that Mr. Clarke has outlined a new assessment of the necessary fire-resistance ratings of reinforced concrete columns in various building occupancies. This is not so, as the paper proposes two sets of simple equations which the author

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†Director, Commonwealth Experimental Building Station, Chatswood, NSW, Australia.

believes, "if the results of further fire tests confirm this theory," could in the future be used to predict the approximate fire endurance of any reinforced concrete column.

Mr. Clarke's paper runs parallel to investigations which he did while employed at the CEBS, and to material published accordingly, but I cannot agree with all that he now has written, mainly because the paper represents an attempted over-simplification of what is a complex problem, and also on the score of practical issues and omissions and the effects which these imply.

The paper draws largely on the work of the Department of Scientific and Industrial Research of Great Britain (Mr. Clarke's Reference 3). Mr. Clarke has analyzed this work in the light of Australian engineering practice, and by a series of approximations, at times hardly justified, he has produced two sets of simple equations, the solution of which requires only a knowledge of the compressive strength of the concrete and the applied load on the column. (*The paper does not take into account differences between Australian and American structural requirements.*)

The applicability of the references from which the author derived his factor of 0.9 for Relationship (4) is, I believe, questionable. Reference 8 is a study of plastic flow under sustained load; the general nature of the statement in Reference 9 is such as to suggest that its author is considering cores taken from mass concrete, rather than a direct relationship between concrete column and concrete cylinder. A figure of 0.8 would be more reasonable, and would yield a factor of approximately 0.7 in Relationship (4a) as against the author's 0.8 and the DSIR figure of 0.65.

For Australian conditions Eq. (2) is not justified for three reasons:

1. Concrete incorporating local aggregates does not exhibit spalling to the extent recorded in Reference 3.
2. The definition of cover in Australian Standard No. CA.2-1958 includes only that concrete between the outermost tie or helix and the concrete surface. In this way Australian covers will exceed British nominal covers by 0.3 in. on average.
3. Recent data made available to the CEBS indicates that the $f_y = 30{,}000$ psi, though to be found in an Australian Standard, is actually very conservative for Australian steels.

When (1) and (2) are considered it would appear that the steel : concrete temperature relationship should be much lower than the 750 C : 400 C which the author assumed. In view of the foregoing I would not agree with the approximation in deducing Eq. (10) from Eq. (9).

The values of "Approximate temperature T of steel prior to collapse, deg C" quoted in Table 1 are not available from the DSIR report (as published) and apparently are "assumed" by the author. This is a major assumption, as the remainder of the table is deduced from these figures.

The values for 15 x 15-in. columns, which are plotted in Fig. 2, establish a reasonable straight-line relationship between the 1½- and 5½-hr ordinates. The four points available for the 19 x 19-in. columns make it difficult to establish the exact location of this relationship, and it appears that the spacing of the intermediate lines for the remaining three column sizes has no justification whatever.

My own application of the first set of equations shows that these equations give conservative values for the fire endurances of small columns, and generous values for large ones. This would be an unfortunate result, if it were applied in practice.

The results quoted by the author for application of the second set of equations amply demonstrate that more knowledge will have to be gained before the problem can be reduced to such simple equations.

The Commonwealth Experimental Building Station recently provided the Standards Association of Australia with a useful direct tabulation of the effects on the fire resistance of reinforced concrete columns of some of the major variables. These include the ratio of the ultimate load-carrying capacity of the concrete to the ultimate load-carrying capacity of the steel, and the ratio of the ultimate load-carrying capacity of the column to the applied load. Because of the paucity of published test results an extension of this task, bringing in also the nature of the aggregate, is proving very difficult.

AUTHOR'S CLOSURE

Mr. Isaacs has had considerable experience in structural design, and for several years he has been engaged in a careful study of all building regulations with a view to assisting Australian legislative authorities to revise existing building codes. However, I feel that Mr. Isaacs has perhaps not realized that the purpose of this paper was to draw attention to the problem by giving a provisional solution and to suggest the basis for further essential research.

The approximations made in this paper are, I believe, in accordance with engineering practice and are appropriate to the variability of the constants associated with concrete design. When further experimental results are forthcoming some refinements can be introduced and some of the approximations can be lessened.

Regarding the factor 0.9 for Relationship (4), I feel that this is fully justified by the references given and this opinion has been strengthened by the recent opinions of some specialists in concrete technology.

I am surprised that Mr. Isaacs dismisses the question of spalling so easily. The investigation on spalling in Australia can be considered to be negligible although I have observed a few instances of spalling with dense concrete. Spalling can occur during fire resistance

tests as many published papers show and had to be allowed for in deducing the compressive strength of hot concrete in this paper.

The value assumed for f_y may, of course, be modified if this theory is to be used with more recent data on mild steel reinforcement. Eq. (10) may be modified in accordance with further experimental results, e.g., it may perhaps be shown that $\phi_s(T) f_y A_s'$ be assumed to be $<240\ P$ or a percentage of 2240 P.

The approximate temperatures of the reinforcement prior to collapse given in Table 1 can be deduced by referring to Reference 3, p. 64, Reference 2, pp. 241-246, or simply from the reasonable assumption that the mean value of the furnace temperature taken from 0 to t hr is approximately the temperature of exposed steelwork at the time t hr. However $\phi_s(T) < 0.3$ when the temperature exceeds 600 C and therefore for these high temperatures an error in estimating the temperature can be disregarded.

The hope was expressed in the paper that further experimental results would soon be forthcoming and until that time the lines drawn in Fig. 2 should be regarded as provisional.

Mr. Isaacs states that the Eq. (17) to (22) give generous estimates for large columns. Perhaps Mr. Isaacs is in possession of fire endurance periods for reinforced concrete columns exceeding 16 x 16 in. which have not yet been published? It is, of course, necessary to have full test data such as the mean compressive strength of cylinders equal in age to the test specimen at the time of testing and this is not given for the 20 x 20-in. reinforced concrete columns described in Reference 2. Furthermore, reloading after subjecting the column to the water test provides little useful data to verify formulas (17) to (22). I feel that even if the estimates should prove to be generous, the true times will exceed at least 3½ hr where Mr. Isaacs obtains possibly 4 hr and (22) gives, say, 5 hr and therefore there should be little cause for concern. In any case, the choice of a safety factor, mentioned in the conclusion, should overcome this objection.

No mention was made in Mr. Isaac's comments of the alternative method of assessing the fire endurance of reinforced concrete columns. It seems to me that more useful and relatively inexpensive research could continued along the lines outlined.

Finally, published work from any source on this subject will be read with interest by those engaged in reducing the present high building costs while preserving an adequate margin of safety.

Discussion of a paper by Robert G. Mathey and David Watstein:

Effect of Tensile Properties of Reinforcement on the Flexural Characteristics of Beams*

By P. W. ABELES, LARRY J. FEESER, HOMER M. HADLEY, K. HAJNAL-KÓNYI, IB FALK JORGENSEN, and AUTHORS

By P. W. ABELES†

This paper is of very great value because it clarifies further the behavior of concrete beams reinforced with well bonded deformed steel bars of high yield strength, and the authors must be congratulated on their clear presentation of the results of their tests. The writer would like to point out a few important results on the basis of his own experience dating back to 1935, when he tested beams reinforced with high strength steel of an ultimate strength of 142 ksi and a yield strength of approaching 100 ksi.[6]

Table 5 is surprising insofar as there appears to be hardly any substantial difference between the results obtained from the straight line and ultimate load theories, the former, in fact, giving results which better agree with the observed values. However, this can be explained by the fact that the authors wisely considered only the effect of the steel, since failure was caused primarily by the steel. However, the table might give the impression that the elastic theory gives better results than the ultimate load theory, which would be incorrect. Different results would have been obtained if the maximum bending moments had been computed for the concrete resistance based on the elastic straight line theory with neglected concrete tensile zone.

The influence upon the behavior of the beams of steel with and without distinct yield strength can be clearly seen from the individual diagrams, (Fig. 6 and 9). This is to be expected for stresses at which the stress-strain curve deviates from the straight line. However, it is surprising that with the beams containing type IV reinforcement a difference in midspan deflection and compression strain occurred at half the yield strength when the stress-strain curve of steel IV appears still to be straight. This apparently is caused by the effect of different E-values of the steels and it is surprising that this is so substantial. A similar effect is also obtained at 50 percent of the yield strength of reinforcing bars VI as compared with reinforcing bars V, but also in this case apparently the strain is greater in the former than in the latter.

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†Member American Concrete Institute, Structural and Civil Engineer, London, England.

These tests give further evidence about the satisfactory behavior of high strength steel well bonded to the concrete even if the concrete strength is relatively low and should be instrumental for the introduction of higher admissible steel stresses. The writer has achieved similar results in his tests mentioned above[6] in which smooth round bars of ⅜- and ½-in. diameter were used and sufficient bond resistance was obtained because the concrete was of very high strength. With dense concrete the cracks of a width of 0.008 to 0.0012 in. (0.2 to 0.3 mm) should be permissible. In fact, tests carried out in 1934-1935 in Vienna on the writer's suggestion[7] proved that with very dense concrete as is obtained by spinning (centrifugal molding), permanently open cracks of 0.012 in. widths are harmless against very severe weather attacks. Further similar tests are being carried out by the writer for the Chief Civil Engineer's Department, British Railways, Eastern Region at which prestressed concrete specimens having cracks of varying widths are exposed to a severe effect of smoke (being placed above the smoke chutes of an engine shed) or placed in the sea between low and high water. These tests which have not been completed, but some of which have extended over five years, have also so far been very satisfactory. Thus no objection should be made to the safe introduction of higher permissible stresses provided that crack widths and deflection are limited. The former can be obtained by good distribution of small size well bonded steel. In this case a much better product can be obtained with regard to cracking (and the danger of corrosion) than with ordinary reinforcement when large-size round bars of little bond efficiency are used. Unfortunately, this latter solution which certainly results in heavy cracks under load is still sometimes employed with full approval of certain authorities which, on the other hand, do not allow the use of prestressed concrete unless, at least in the calculation, the exclusion of tensile stresses in the concrete under working load is avoided.

While the size of cracks can be satisfactorily limited to the permissible extent even with stresses of 50,000 psi (as stated before by the provision of well distributed and well bonded steel) the deflection might become excessive. This led the writer to suggest partial prestressing in this Journal in 1945,[8] so as to have the extent of cracking and deflection controlled by the designer and the amount of reinforcement appreciably reduced by allowing average safe working load stresses up to 100,000 psi.

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By LARRY J. FEESER*

The authors have added valuable information concerning the action of beams reinforced with high tensile steel.

Once again, as in Clark's tests[2,9] compressive strains were observed on the tension side of the beams tested, but no attempt was made to explain why these compressive strains exist. It would be interesting to know what stresses existed in the tensile steel just prior to testing. Since no data were obtained on this in the authors' tests, similar tests with more complete taking of data can furnish a clue as to the reason for these compressive strains.

Rensaa[10] surmised that tensile shrinkage strains in concrete could be of the order of 300 micro-in. per in. when restrained by 1.5 percent steel producing 7533 psi compression in the steel. By use of stable, long-time bakelite electric strain gages, in which he took zero readings just prior to placing of the concrete, Rouse[11] recorded stresses as high as 8000 psi compression in the flexural steel of unloaded beams after curing. These stresses were recorded, in most cases, from two to four weeks after placing of the concrete. This would indicate that there existed in the concrete surrounding the steel, shrinkage strains as high as 270 micro-in. per in. tension.

It seems logical to assume that shrinkage effects, similar in nature if not in magnitude, were present in the authors' beams at the time of testing. This being the case, it is not difficult to see how compressive strains in the concrete could occur on the tension side of the beams. When cracking first starts, the stresses due to shrinkage in the concrete are partially relieved, which allows some compressive rebound in the concrete between cracks. As the cracks progress toward the compression side of the member, and as some bond slip occurs at the cracks, the shrinkage stresses are further relieved and the compressive rebound becomes greater, thus explaining the increase in compressive strains with increase in load.

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*Member American Concrete Institute, Instructor of Civil Engineering, University of Colorado, Boulder, Colo.

By HOMER M. HADLEY*

This paper is a valuable one. In a sense it is only a demonstration of the obvious: reduced steel areas, increased unit stresses, and unaltered modulus of elasticity inevitably result in wider cracks and greater deflections. But this series of tests gives precise quantitative values and is thorough, comprehensive, and reliable. Data of such character are always valuable. Its authors are to be thanked and congratulated upon their work.

The Type IV bars appear to have little to commend them. While presenting more bearing area per linear inch than any other bars, their modulus of elasticity was 10 percent lower than that of the other bars. Two #4 and one #3 bars, bundled, would have provided practically the same cross-sectional area, a greater bearing surface per linear inch, and in general would have acquitted themselves better than the Type IV bars did. Another time it would be well to use them.

Fig. 5 presents deflections in relation to applied loads and to computed deflections. Even at extreme loads the agreement between observed deflections and deflections computed by the modified straight-line theory is remarkably close. With loads less than 80 percent of maximum values the agreement between deflections calculated by straight-line theory and observed deflections is quite close. Since it is to be hoped that loads applied to structures will never be so great as to approach their ultimate capacity of resistance, there is satisfaction to be found in this proven agreement between theory and actuality.

Perhaps most interesting of all findings of this paper is recorded in Table 5. This tabulation shows that moments computed by the scorned and much-despised straight-line theory more closely agreed with the actual observed moments than did those computed by ultimate strength theory! How is it possible that such an amazing thing can be? Was it not claimed both volubly and emphatically, that the ultimate-strength theory was a positive improvement on the straight-line theory, rather than a negative improvement on it? Yet how are these claims to be reconciled with Table 5? And if they can't be reconciled, what is the point, the raison d'etre, for the ultimate-strength theory? If it is supposed to approach more closely to the reality of nature, why does it not approach, rather than depart more widely from it?

It is all the more bewildering because the June 1960 issue of *Concrete Construction* contains an article, "Revolution in Concrete," predicting a vast expansion in the structural uses of concrete based on (1) higher strength steel and high quality concrete, (2) "Exciting design concepts known as ultimate strength and limit design," (3) "Narrowing the 'area of ignorance' known as the safety factor in concrete construction."

*Member American Concrete Institute, Consulting Engineer, Seattle, Wash.

Almost involuntarily one finds oneself thrilled and enthused! Almost involuntarily he tends to cheer and to hurrah! And yet this paper says use high strength steel only at the price of increased deflections. Regarding "ultimate design" and, respecting "areas of ignorance," it makes no comment, unless it be mutely and inarticulately.

It is all most bewildering.

By K. HAJNAL-KÓNYI*

The paper is an important contribution to the establishment of an upper limit of the yield or proof stress of steel up to which high tensile steel can be utilized as tensile reinforcement in concrete on the basis of the equation $A_s f_y = A_s' f_y'$ without a reduction of the permissible moment, where A_s and A_s' denote the sectional areas, f_y and f_y' the yield strengths of mild steel and high tensile steel, respectively. The writer has investigated this question in several papers.[12,13]

In structures with predominantly static loading the problem has to be considered from the points of view of (1) ultimate load (2) cracking (3) deformation.

ULTIMATE LOAD

The above equation is only valid for under-reinforced sections. With the standard method of design the permissible moment decreases with increasing working stress in the steel even in the case of under-reinforced beams, if the permissible concrete strength is fully utilized, whereas with the ultimate strength theory only the various coefficients at "balanced design" have to be checked. These depend on the critical strain in the concrete and in the steel at failure (ε_c and ε_s). For a given quality of concrete ε_c is constant, whereas ε_s increases with increasing yield strength of the steel and causes a reduction of the depth of the neutral axis and of the relevant coefficients for balanced design. With the qualities of materials used in the tests described in the paper, Beams I to IV were obviously underreinforced, whereas Beams V and VI require a closer investigation. For the sake of simplicity, the following calculations are based on the averages of the two beams in each group. The numerical values and notations used are from Reference 12.

α = ratio of resultant concrete compression C to $n'bdf_c'$
k = factor determining the distance of the resultant of the assumed rectangular stress block $qbdf_c'$ from the edge of the concrete.

q = tensile reinforcement index = $p \dfrac{f_y}{f_c'}$

For $f_c' = 3725$ psi, $\varepsilon_c = 0.0034$, $\alpha = 0.756$,

$k = 0.60$ (see Reference 12)

*Member American Concrete Institute, Consulting Engineer, London, England.

$$\varepsilon_s \text{ for Type V} = \frac{98{,}800}{28{,}100.00} + 0.002 = 0.0055$$

$$\varepsilon_s \text{ for Type VI} = \frac{104{,}300}{28{,}300{,}000} + 0.002 = 0.0057$$

For balanced design (index b):

$$n_b' = \frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s}, \quad q_b = \alpha n_b'$$

	Beams V	Beams VI
n_b'	$\frac{0.0034}{0.0034 \times 0.0055} = 0.382$	$\frac{0.0034}{0.0034 \times 0.0057} = 0.374$
q_b	$0.756 \times 0.382 = 0.289$	$0.756 \times 0.374 = 0.283$
q_{actual}	$0.0112 \times \frac{98800}{3720} = 0.297$	$0.0113 \frac{104,300}{3730} = 0.316$
q_{actual} / q_b	1.027	1.12

It may be seen from the last line that Beams V were as nearly "balanced" as can be achieved in laboratory tests, whereas theoretically Beams VI were slightly overreinforced. It has been pointed out in Reference 12 that the method adopted in working out the coefficients for "balanced design" is too conservative in the case of deformed bars since it does not take into account the uneven distribution of strain along the bars which takes place as long as the bond between cracks is not destroyed. Consequently in cases like Beams V and VI it has to be expected that the yield stress can be reached at failure. With this assumption we obtain

Beams V

$$M_{ult} = 0.297(1 - 0.6 \times 0.297)12.8^2 \times 6 \times 3720$$
$$= 891{,}000 \text{ in.-lb}$$
$$M_e \quad 940{,}000 \text{ in.-lb}$$
$$M_e/M_{ult} \quad 1.055$$

Beams VI

$$M_{ult} = 0.316(1 - 0.6 \times 0.316)12.8^2 \times 6 \times 3730$$
$$= 938{,}000 \text{ in.-lb}$$
$$M_e \quad 969{,}000 \text{ in.-lb}$$
$$M_e/M_{ult} \quad 1.032$$

The results confirm the expectation since the theoretical values have been exceeded in both groups. The excess is smaller in Group VI than in Group V, as was also to be expected. In such cases it is very difficult to distinguish a compression failure from a tension failure so that probably the mode of failure was uncertain in all four beams, not only in the two where it is so described. In the case of Beams V the ratio of the maximum bending moments was 1.017 as against a ratio of the cylinder strengths of 1.05 so that if failure of the beam with weaker concrete was "uncertain" one would not expect the beam with stronger concrete to fail in compression. In the case of

Beams VI the cylinder strengths were equal, yet according to Table 4 failure in compression occurred with the beam which carried 3.8 percent less load than the beam where failure was "uncertain."

The uneven distribution of the strain along the bars (i.e., high concentration within the cracks and reduction between the cracks where the bond is still effective) is confirmed for Beams VI by Fig. 7. In Beams I the bond was not so good because of the large sizes of the bars and in Beams IV probably because of their shape.

It is interesting to compare the calculated position of the neutral axis in Beams VI with the value derived from strain readings from Fig. 1, 7, and 8. At the highest load for which readings are recorded, the steel stress converted from strain was approximately 80,000 psi (Fig. 7). The corresponding strain according to Fig. 1 must have been approximately 0.0034. The strain at the face of the concrete was approximately 0.0026. Hence $n' = 0.0026/0.0060 = 0.43$. In a beam which is nearly balanced the position of the neutral axis remains practically constant near failure. The theoretical value, corresponding to q_{actual} is $0.316/0.756 = 0.42$. The agreement is better than could be expected by scaling from the small diagrams in the paper.

The percentage of reinforcement in Beams I corresponds to a percentage of $2.55 \times 42{,}500/40{,}000 = 2.71$ percent with steel of a yield point of 40,000 psi. According to the ACI 1956 Code the maximum percentage of such steel in concrete of $f_c' = 3750$ psi without compression steel is 3.75 percent, resulting in $M_{ult}/bd^2 = 1147.5$ psi. With a load factor of 1.8 the maximum permissible value of M/bd^2 without compression steel is 638 psi. In contrast the British Standard Code of Practice C.P. 114 (1957) limits the latter value to 1/4 of the permissible working stress = 1/12 of the cube strength at 28 days. Concrete of $f_c' = 3750$ psi corresponds to concrete of a cube strength of 4500 psi with an upper limit of $M/bd^2 = 375$ psi and a corresponding percentage of 2.5 percent of steel with $f_y = 40{,}000$ psi.

It follows from the preceding that on the basis of the tests discussed, steel with f_y up to 105,000 psi could be used instead of steel with $f_y = 40{,}000$ psi at percentages in the inverse ratio of their respective yield points indiscriminately if one adheres to the British Code, as far as the ultimate moment is concerned. This, however, is not true if the design is made in accordance with the ACI 1956 Code. It is safe to conclude from the tests that beams with steel of $f_y = 105{,}000$ psi and ¾-in. diameter in 3750 psi concrete with a percentage of $3.75 \times 40{,}000/105{,}000 = 1.43$ percent ($q = 0.400 = 1.415q_b$) would fail in compression at a steel stress $< f_y$. The limit $p = 0.4f_c'/f_y$ in the ACI Code has to be reduced with increasing f_y. Numerical values have yet to be determined by tests.

It should be added that the diameter of the bars has an influence on the limiting values of q and p. In the writer's test Beam 23H (Ref-

erence 12) the proof stress has been reached even with $q/q_b = 1.49$ because wires of 0.104-in. diameter were used.

CRACKING

The preceding considerations are based on ultimate load only, i.e., the requirement of safety against collapse. It is of equal importance that the structure should not become unserviceable at the design load because of excessive cracking.

The limiting crack width depends on the use and the exposure of the structure. The views of various experts on this question vary within wide limits[14] but a limit of 0.30 mm = 0.012 in. in "indoor" structures seems to be widely accepted.

It may be seen from Fig. 12 that both in Beams V and VI this limiting crack width was observed at a calculated steel stress of about 38,000 psi. This is less than the corresponding stress in Beams II, III, and IV where it was 42,000 to 44,000 psi. The less favorable behavior of Beams V and VI can be explained partly from Clarke's formula which contains the factors $(h - d)/d$ and D/p. The factor D/p was greater in Beams V and VI than in the others. Further, the factor $(h - d)/d$ is a simplification. In the formula $h - d$ is the distance of the center of gravity of the reinforcement from the face, whereas the relevant value is the cover, i.e., the distance of the surface of the bars from the face of the concrete. In Beams I this was 1.64 in. whereas in Beams V and VI it was 1.83 in. The substitution bar for bar at the same effective depth was, of course, correct in the tests for the purpose of comparing ultimate loads. However, in practical cases, except for extreme fire hazard, such a cover is excessive for ¾-in. diameter bars. Moreover, a good designer would provide three bars of ⅝-in. diameter instead of two bars of ¾-in. diameter of high tensile strength in a 6 in. wide beam. This would reduce the crack widths.

In this connection, the authors' previous contribution[15] to the question of crack widths should be mentioned. According to their test results[15] the crack width at the surface of deformed bars does not increase in linear relation with the crack width on the surface.

In view of all these points, it is justified to conclude that in indoor structures *with good detailing* working stresses of the order of 40,000 psi can safely be permitted from the point of view of crack control if the fire hazard is not abnormally high. If the diameter of the bars is limited to say ½ in. which would not be unreasonable with steel having a minimum yield point of 90,000 psi and over, stresses of the order of say 50,000 psi would still be harmless. A further increase of the working stress would necessitate a further reduction of the diameters.

For "outdoor" structures under normal conditions the limiting crack width agreed by the majority of experts is 0.20 mm = 0.008 in. It may

be seen from Fig. 12 that the corresponding steel stress is approximately 30,000 psi. Thus, on the basis of a factor of safety of 1.8, as specified in the ACI Code, the economic limit of f_y would be 54,000 psi, i.e., only slightly above that of "hard grade" steel. The factor of safety required by the British Code is 2.0, corresponding to $f_y = 60{,}000$ psi which is the specified minimum proof stress for cold worked steel according to B.S. 1144.

With small size bars, well distributed in the cross section of the concrete for which various formulae are given in European Codes, stresses higher than 30,000 psi could be permitted in a similar way as the stress of 40,000 psi could be exceeded in "indoor" structures.

It should be pointed out that for the purpose of preventing corrosion the density of the concrete is of far greater importance than the crack width on the surface. Corrosion can occur in concrete which has not been well compacted even without any cracks as exemplified by fence posts.

DEFORMATION

The authors have found that at 50 percent of the nominal yield strength, the deflection of Beams VI was 2.3 times that of the deflection of Beams I. This is roughly the inverse proportion of the percentages of reinforcement, as was to be expected. However, the deformation due to dead load can be more or less compensated by an appropriate camber so that in structures where the dead load is a substantial proportion of the total load (e.g., roofs), the greater deflection in short time loading may not be a disadvantage.

More important is the additional deflection caused by shrinkage and creep. This is much less affected by the steel stress than the initial deflection as was shown in Reference 15, Fig. B, although the influence of the higher steel stress is not negligible. Consequently, higher working stresses in the steel may necessitate in certain cases an increase of the depth of slabs and beams so as to avoid excessive deflections.

For a given steel stress the deflection increases with increasing percentage. This may be seen from Fig. 5 by comparing the deflection of Beams VI at say 10,000 lb with that of Beams I at 23,000 lb. The ratio long time deflection/instantaneous deflection also increases with increasing percentage. The limitation of the deflection is one of the reasons why it is not advisable to approach the limit $p = 0.4 f_c'/f_y$ even with mild steel.

SUMMARY

1. The authors' tests confirm that steel with $f_y = 105{,}000$ psi can be substituted for steel of lower f_y in the inverse proportion of the respective yield strengths without loss of load bearing capacity as long as the limit of "balanced design" is not exceeded. According to the

British Code of 1957 this substitution is possible up to the maximum percentage of mild steel which is permissible without compression steel. If this limit is exceeded, there is a risk that the beam with f_y = 105,000 psi becomes "overreinforced." This is to be expected with certainty if the limit in the ACI 1956 Code is approached. In the writer's view, the limit of $p = 0.4 f_c'/f_y$ is too high even for mild steel because it may result in excessive slenderness of concrete members.

2. With a factor of safety of 1.8, the working stress would be 58,000 psi. The limit for indoor structures seems to be 40,000 psi for large size bars with a possible increase to 50,000 psi with bars not exceeding say ½-in. diameter. A stress of 58,000 psi could only be permitted with very small diameters, say ¼ in.

3. The limitation of the deflections necessitates in certain cases either a reduction of the stresses under (2) or an increase of the depth of concrete sections. This means in other words that the depth : span ratio has to be greater than for mild steel.

4. It would appear, therefore, that for general use in indoor structures f_y = 72,000 to 80,000 psi is the economic limit. Steel of higher f_y can be used at its full strength only with certain restrictions.

5. For outdoor structures under normal conditions the economic limit is f_y = 54,000 to 60,000 psi.

6. An increase of f_y beyond 105,000 psi can hardly be economical for nonprestressed reinforcement even with very small diameters.

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By IB FALK JORGENSEN*

The test series has been executed very diligently and the results have been analyzed in a very commendable way. However, it is my opinion that the scope of the work and the purpose for the series is

*Member, American Concrete Institute; Ib Falk Jorgensen, Consulting Engineers, Inc., Denver, Colo.

not satisfactory considering the fact that the test specimens were reinforced with a steel percentage twice that which generally gives the economical optimum for reinforced members. This percentage is about 60-80 percent of balanced reinforcement as defined below.

With $f_c' = 3650$ psi, and $f_s = 20{,}000$ psi, a balanced reinforcement requires a steel area of 1.25 sq in. for the beams in question (which has an effective depth of 12.8 in. and a width of 6 in.). To make the test series realistic and applicable to general design practice the steel areas used should be about 1 sq in. for the 40 ksi yield steel. For beam Type I, the area of reinforcement is approximately double this amount. For all beams involved, Type I through VI, the steel area is in excess of balanced reinforcing as defined by the straight-line theory. However, in comparison to the maximum steel area allowed under the ultimate strength design method, the steel areas are less than the maximum as allowed by code. The results do not compare too well with previous tests and findings with lower steel percentages, probably because of the high steel percentage employed.

The low modulus of elasticity exhibited by type number IV bars is readily explainable from the shape of the bar. The major mass of the bar is in each dumb-bell, and the major forces absorbed in the bar have to travel along the helical shape of each part of the dumb-bell, hence the length of travel for the force is a few percent longer than if it travels straight along the axis of the bar as is the case for a standard rebar. In addition to this, the smaller amount of material between the two dumb-bell shapes has a greater influence on a cross sectional strain related to Poisson's ratio. By evaluating these factors it is easy to understand that the modulus of elasticity for the Type IV bars is several percent less than that of an ordinary bar.

The size and shape of the test specimen is very commendable; however, the bonded wire strain gages were only applied in the middle of the span. Strain gages should have been placed at frequent intervals of the full 10 ft length of reinforcing bars. The same should have been done for the wire strain gages for measurement of compressive strain. Also, the Tuckerman strain gages should have been placed over the complete length of the beam. The reason for these recommendations is that for a test series with such high steel percentages, the test specimen under failure loading or close to failure loading will act like a tied supported arch. By the term tied supported arch, it is meant that an arch forms in the concrete within the beam and the reinforcing acts like the tie rod which again supports the arch through the concrete columns formed between cracks. This tendency is fairly well shown on Fig. 4 where the crack formation extends into the shear areas. This specific effect of a tied supported arch is favored by the fact that all the reinforcing bars were anchored thoroughly to a plate at each end of the beam. If the gages had been placed as mentioned

previously, the amount of this effect could have been verified. As the test series was executed, we have no way of knowing the influence of the tied supported arch effect upon the results. One reference to this effect is a paper by the Authors.[17] Fig. 6 in this paper shows that at loads close to the ultimate the steel strains in the reinforcement 3 in. from supports were within a few percent of the strains in the reinforcement at midspan. A second reference is a Danish publication.[18] The tied supported arch effect may very well be the reason why the report does not correspond with a number of previous findings.

The observed moment relative to the ultimate moment of Type IV beams is contrary to previous tests, for steels which do not exhibit natural yield point. In general, the observed moment would have been 5 to 15 percent greater than the moment calculated by ultimate design using nominal yield strength. Numerous tests have been conducted at the Royal Danish Institute of Polytechnic and at the Danish Naval Testing Station showing the higher moment. These results have prompted the Danish building code to allow higher working stresses for rebars not displaying a natural yield point. Hajnal-Kónyi has parallel findings.[19] The test series described in his report had lower steel percentage than this test series. Hajnal-Kónyi found that the observed moment was about 10 percent over and above the calculated ultimate moment. The reason for not obtaining this result can be explained by the lower modulus of elasticity for Type IV bars which contributes to a greater rotation of the concrete sections around a crack. This, along with the high steel percentages, increases the concrete stresses up to the point of failure.

The effect of lower modulus of elasticity of Type IV bars on the width of cracks is less than might have been anticipated. This presumably is caused by the relative greater bonding capacity in the steel as selected compared to the bonding capacity of the bars in Type II and Type III beams.

Assuming that half of the ultimate carrying capacity of the beam is the working load level, none of the specimens show an average crack width in excess of 0.010 in. which is very desirable from an erosion and rust protection standpoint. However, from a standpoint of warning, the beams of Type IV, V and VI display greater increase in maximum crack width at loadings above working level and, therefore, a superior warning before failure. The crack width is such that it is very visible and the size of it does not effect the durability of the structure from an erosion standpoint as it only occurs at a time close to failure.

Also, Beams IV, V and VI show increased deflection above the working levels. This nonlinear increase of deflection above the working level is also a good exhibit giving an additional warning.

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AUTHORS' CLOSURE

The authors appreciate the contributions and interest shown in the paper. These discussions were not only valuable additions to the paper but also clarified some of the questions which occurred to the readers.

In regard to Mr. Hadley's discussion pertaining to the agreement between observed and computed maximum moments based on the straight-line and ultimate-strength theories, the authors wish to cite a recent publication by Rusch.[20] Dr. Rusch states that "The agreement between results of various theories in design, however, is not at all surprising since only the case of under-reinforced beams has often been cited in comparison with test data. The tensile force in the steel at failure is determined entirely by the yield point; the lever arm of the internal forces is insensitive to assumptions regarding concrete stress." Thus, it is not at all unexpected that the maximum moment can be accurately predicted by the straight-line theory for beams which fail in tension. This view is also supported by Dr. Abeles' statement that the good agreement between maximum bending moments computed by the straight-line theory and the observed values can be explained by the fact that failure was caused primarily by yielding of the steel. The authors agree emphatically with Dr. Abeles that had the maximum bending moments based on compressive strength of concrete been computed by the two theories, the agreement between observed and computed values would have been better for the ultimate-strength design theory.

Mr. Jorgensen pointed out correctly that the beams were over reinforced as defined by the straight-line theory. It will be recalled that the factor of safety against compression failures in this theory is high, thus restricting the ratio of reinforcement to relatively low values. In this respect, the ultimate-strength theory provides for more realistic values of balanced reinforcement. The control beams in the investigation were designed to assure failure by yielding of the tensile reinforcement. These beams were reinforced with intermediate-grade bars (Type I) and the ratio of reinforcement was about 75 percent of the maximum value specified in the ultimate strength theory. Thus, the authors considered these beams to be reinforced with an amount of steel which might be used by designers utilizing the ultimate-strength design method.

The authors wish to point out that the test results in general compared favorably with previous tests. For example, the average width of cracks were predicted with reasonable accuracy by A. P. Clark's formula. It is recalled that Clark used intermediate-grade bars in his test program. The maximum moments computed by both the straight-line and ultimate-strength theories agreed reasonably well with observed values, and the midspan deflections were fairly accurately predicted for beams containing bars having a nearly linear stress-strain relationship.

The authors appreciate Mr. Jorgensen's desire to have bonded wire strain gages placed on the reinforcing bars at frequent intervals along the span length. However, this would have affected the random occurrence of cracks and would have led to erroneous values of average width and spacing of cracks which were important variables in the investigation. The average width of cracks were determined in the region of constant bending moment in order for the data to be applicable to other beam tests.

The authors agree with Mr. Jorgensen's statement that the beam specimens exhibited some tied arch action as the loads approached their maximum value. However, the authors do not believe that the tied arch action in their beam specimens was present to the same extent as in the beams described in the paper "Strains in Beams Having Diagonal Cracks." The beams in the present study were heavily reinforced with stirrups whereas the specimens in the paper quoted by Mr. Jorgensen contained no web reinforcement and developed sizeable inclined cracks which extended well into compressive zone at loads equal approximately to one-half the ultimate load.

Mr. Feeser commented on the presence of compressive strain on the tensile face of the beam as reported by the authors and offered the theory that compressive strains develop as shrinkage strains in the concrete are relieved by formation of cracks. The authors offered another explanation given in Reference 2 of their paper. In the paper to which the authors referred it was demonstrated that the surface of the concrete at a cracked section "dishes out" as the steel stress increases causing the exterior surface to contract. This phenomenon was observed in tensile bond specimens which simulate the behavior of a segment of a beam between two successive cracks.

The authors appreciate Dr. Hajnal-Kónyi's comprehensive discussion which presents a clear analysis of the maximum steel stresses in the tensile reinforcement which can be safely used in design.

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Discussion of a paper by W. F. Cole and B. Kroone:

Carbon Dioxide in Hydrated Portland Cement*

By R. GAZE and ROBERT H. S. ROBERTSON, M. SPINDEL, A. STEOPOE, and AUTHORS

By R. GAZE and ROBERT H. S. ROBERTSON†

Messrs. Cole and Kroone have certainly unraveled the mystery of the crystal forms of calcium carbonate which occur in hydrated portland cement and in calcium silicate hydrate, adding great clarity to our earlier observations. Doubtless further work will establish the rate of carbonation and will relate this effect to the specific surface; the indications so far are that the calcic outer surface of the crystals is attacked monomolecularly at first; and carbonation of calcium ions lying further inside the lattice takes place much more slowly. It follows from this that the calcium carbonate arising from a two-dimensional source is at first in very small crystallites. We have not been able to detect these on electronmicrographs of our preparations, even at magnifications up to 100,000 ×. In shadowed crystals some are smoother than others, but many more pictures would be needed than we have to see if exactly one half of the crystals can be observed "butter side up." It would be interesting to know if the $CaCO_3$ crystallites are topomorphic or at least epitactic; electron diffraction might settle the point.

By M. SPINDEL‡

This paper gives useful information, apparently as a supplement to a previous paper by Kroone and Blakey.[12] The writer may be allowed to add some information on this matter, part of which was published before the references given by the authors.

The effect of carbon dioxide on hydrated lime was known since the antique ages to be the cause of hardening and giving strength to lime mortars used in buildings and various structures. In this century the carbon dioxide was also known to increase the strength of mortars and concrete made of portland cement and also of lime silica bricks made

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†The Cape Asbestos Co., Ltd., London, England.

‡Member American Concrete Institute, Research Engineer and Consultant, London, England.

by autoclaving hydrated lime and silica. In some countries in Europe tests for portland cement were standardized for which the specimens of cement mortar after 24 hr in the molds were cured for the first 6 days in water and then for 3 weeks in dry air, thus unintentionally reaching much higher strengths than concrete or mortar in specimens of larger size or on the job in usual circumstances.

As soon as the writer equipped the first laboratory for testing materials at the Austrian State Railways in 1907 he started to test the progress of carbonation of cement mortar and also of lime silica bricks, simply by placing some drops of phenolphthalein dissolved in alcohol on broken specimens. Thus the progress of carbonation could easily be seen in every fresh broken sample from time to time. These tests even succeeded in checking the age of steamed lime sand brickets stored and cured in the air on the job.

The disadvantage of using phenolphthalein solutions was that the red color of the still alkaline parts of the sample disappeared after some time and was thus not quite suitable for later comparisons.

Lea and Desch[6] mention several tests to be used for testing with various dyes the color of the different chemical compounds in hardened cement and mortar, including carbonated lime. It is surprising that the author did not mention any of these tests. The writer during the last war and later used successfully Nigrosine Stain combined with phenolphthalein for showing how far the carbonation of hydrated lime took place, and this test gave the most stable and lasting effect of black color for carbonates on the specimens tested.

In 1909 an important contribution to the problem of carbonation of hydrated lime was made by Nussbaum.[34] This book states that the author, professor of hygiene at the University of Hannover, and his colleague, Dr. K. B. Lehmann, professor of hygiene at the University of Würzburg, carried out special tests with regard to carbonation of hydrated lime in mortars. The essentials of these investigations still worth considering were:

1. The hardening of lime hydrate in mortars by carbonation goes on slowly. If saturated with water the carbon dioxide can reach only the surface of the mortar, forming there a thin film, and if only 0.5 volume percent of water is left in the mortar, the hardening process stops completely. A slow, hardly observable, hardening takes place between 0.5 to 1 volume percent of water and also when, after water saturation it begins to dry, which, according to the mix proportion, may still be between 8 to 10 volume percent of water. The effect of carbon dioxide is satisfactory between 1.5 and about 6 volume percent and lively between 2 to 5 percent of water by volume of mortar. Apart from that, the air must be in a position to surround the mortar completely to be successful with the carbonation of lime. This occurs easily with porous mortars, but not with watertight concrete.

2. For these reasons and based on their investigations also in practice the writers claimed that only the lime rendering on the outside of walls

of houses was really hardened by carbonation of lime, but the rendering of the inside walls was only left a dry powder beneath a film of hardened carbonate of lime. They were also of the opinion that the strength of lime in old houses and structures was due to various pozzolans used together with lime and not to lime alone.

The above information is given to the authors of the paper under discussion with the request to state whether the lower and upper limits of water for carbonation are in accordance with their experience, as it was approximately with the experience of the writer.

On just one more point the writer would like to have the opinion of the authors: To find the effect of carbonation, even as far as this process does not continue until the concrete is completely destroyed by conversion into easily soluble double carbonate, it is first of all necessary to know the chemical composition of the various hydrates in the hydrated cements tested by the authors. We have known for some time the nearly exact chemical composition of the different particles of dry cement powder, but we also know that there are still doubts left about the exact composition of their various hydrates. Especially the lime-silica hydrates are known to vary with regard to the ratio of lime-silica and the amount of chemically combined water of the various lime-silica compounds in the hardened cement. This might perhaps help to clarify the position of the various different carbonates found by the authors with the latest methods of investigation.

Especially concerning the hydrated calcium silicates, the writer[35] gave information on the development and state of affairs regarding the clinker minerals of portland cement and their hydrates. Fig. 3 shows the several different hydrates of the calcium silicates, especially with regard to the hydrates obtained as sediments after shaking and dissolving the most important minerals of portland cement clinker in a large quantity of water by Nacken.[36]

The main conclusions the writer then drew from these tests, which are still valid, is

1. that the quantity of lime in hardened concrete depends not only on the chemical composition of the cement but also on the quantity of water added.

2. That as long as some free lime hydrate in concrete is still left, no aggression of pure water on concrete will occur, and therefore it might be useful to have some left not carbonated.

3. With regard to aggressive water, the mortar and concrete being waterproof is more important than a small difference in the chemical composition of the cement.

To this it may be added that surface hardening of concrete could be achieved much more effectively than by carbonation by using chemicals which combine with the hardened concrete to make it insoluble in pure and also in mildly aggressive water. Further, it should be mentioned that there are certain admixtures available to

make the concrete waterproof and inpermeable thus avoiding the hydrated lime being washed out by percolating water and also the deterioration of concrete by the so-called aggressive carbon dioxide and other aggressives.

How much the different types of carbonate, i.e., vaterite, aragonite, and calcite are found separately in the concrete after carbonation and the conclusions based on it by the authors is, of course, useful to know from the scientific point of view, but we may suppose that all three types of carbonates will behave in a similar way regarding their effect on strengths and resistance to aggressives.

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By A. STEOPOE*

The corrosive actions exerted on hardened cement must be considered as reactions in heterogeneous systems. In a fluid-solid system, all these reactions begin with adsorption and chemisorption processes and not with the formation of a definite and well crystallized compound.

The hardened cement represents a complex system of gels and crystals of different chemical compositions, in which the main ionic component is Ca^{2+}. This ion is bound in different manners, with variable bound-energies. So, it can be adsorbed or chemisorbed on the surface of the hydrated compounds, or bound in $Ca(OH)_2$ crystals, or in the lattices of the crystallized compounds as lightly hydrolyzable ion, which can be removed not only by the action of the water, but also through base exchange reactions.

In such a complex system, I believe that the mechanism of the CO_2 action can be explained as follows:

All the hydrated compounds of the hardened cement are surrounded by adsorbed water films formed of movable H_2O-molecules, which differ in their properties from those bound in the lattices of the crystallized and hydrated compounds. The CO_2 action occurs in these water films, in which the gas is soluble. Firstly, the CO_2 acts on the most weakly bound Ca ions, that is to say on the adsorbed ions, to form isolated $CaCO_3$ molecules, which are not recognizable by x-ray diffraction, because they do not form crystals initially but are finely disseminated. Then the CO_2 action extends on the hydrolyzable and zeolitically bound Ca ions and finally on those bound in crystalline lattices.

*Bucharest, Rumania.

At the same time with the progression of the carbonation, the content of the hydrated compounds in siliceous gels increases.[3] Or it is known that these gels adsorb the CO_2 gas proportionally with its pressure.[37] On heating, the adsorbed gas is liberated gradually at temperatures lower than those necessary for the dissociation of the carbonates present in the hardened and carbonated cement.

I propose this explanation for the experimental observations reported by Messrs. Cole and Kroone.

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AUTHORS' CLOSURE

We are interested to learn from Dr. Gaze and Mr. Robertson of the difficulty that they have experienced in detecting crystals of calcium carbonate underneath the electron microscope even at magnifications of 100,000 ×. However, the electron diffraction technique as we have indicated in our reply to Dr.Steopoe's comments and as Dr. Gaze and Mr. Robertson suggest, could well show the presence of calcium carbonate crystals and the manner in which they have grown. Unfortunately this technique is not available to us.

We wish to thank Mr. Spindel for bringing to our attention his early paper in *Zement*[35] and the earlier book in 1909 by Nussbaum[34] which deals with the problem of the carbonation of hydrated lime. Unfortunately these publications are not available in Australia. However, our paper does not concern itself with the action of carbon dioxide on hydrated lime but deals with its action on cement mortars and calcium silicate hydrate. We believe that this reaction takes place in the following way: Hydrated cement minerals → siliceous residue + calcium hydroxide → vaterite + aragonite + poorly crystallized calcite → well-crystallized calcite. We note, however, that when the CaO formed from calcined calcite is allowed to carbonate in air it first hydrates to form calcium hydroxide which is converted to carbonate by a sequence of events similar to those involved in the carbonation of hydrated cement minerals as set out above; a fact which is not well known.

Although phenolphthalein has been used in these laboratories to indicate the presence of calcium hydroxide in stored cement samples there would have been no point in using such an indirect method of determining the mineral composition of our mortars and calcium silicate hydrate samples when x-ray diffraction facilities were available.

In our paper we gave the composition of the hydrated cement minerals present in both the mortar and the calcium silicate hydrate samples

but noted that it is difficult to recognize hydrated cement minerals in (a) set mortars after short curing times or (b) calcium silicate hydrate after short autoclaving times; in our samples the mineral compositions were much the same. Nevertheless, it is possible that a difference in mineral composition that we could not detect determined the dominance of vaterite over aragonite in the mortar samples, and the dominance of aragonite over vaterite in the calcium silicate hydrate samples, as Mr. Spindel suggests.

If we assume that mortar has a density of 2.5 it is possible to show that carbonation of lime mortar takes place over a range of water content of 0.8-2 percent on the values quoted by Mr. Spindel from the book by Nussbaum. These figures are to be compared with a range of water content of 0.5-3 percent for the carbonation of cement mortars from our work. The agreement is very close.

We agree with Dr. Steopoe that hardened cement consists of a complex system of gels and crystals of different chemical compositions with Ca^{2+} as the dominant cation held in these structures with different bond energies. We note though (Bogue,[29] p. 644) that material from hydrated $3CaO \cdot SiO_2$ that is gel-like beneath the electron microscope gives crystalline electron diffraction patterns. That the Ca^{2+} ion is adsorbed or chemosorbed on the surface of the gels and that it can be moved from the gels, $Ca(OH)_2$ crystals and crystalline hydrated compounds by base exchange reactions we rather doubt, although its migration in solution seems to be accepted (Bogue, p. 656). We also agree with Dr. Steopoe that the CO_2 reaction takes place through the films of CO_2-containing water that surround the hydration compounds. However, that the reaction proceeds as Dr. Steopoe suggests is conjectural. Our work has indicated that the $CaCO_3$ materials that are formed are by no means isolated but are linked together to form a definite yet poorly crystallized three-dimensional lattice. The crystal structure of the carbonate minerals indicates that they could hardly crystallize in any other way. Electron diffraction patterns of calcium silicate hydrate minerals have been reported to be dominated by the sharp lines of calcite due to surface conversion to carbonate (Bogue, p. 635). Since the calcite disappears when the bulk of the carbon dioxide is lost on heating, and larnite appears from a reaction between the fired CaO and the adjacent siliceous residue, we see no reason to postulate that the carbon dioxide is adsorbed as a gas in the siliceous residue.

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